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IN THIS ISSUE:

Arch Dam of Ice Stops Landslide

Stage Construction of St. Louis Streets

Grouting Backfill Under Paving Cuts

Drilled Holes For Bridge Piles

***Meeting Reports*:**
Asphalt Technologists
Brick Paving

•

MISSISSIPPI FLOODS

Staff Report of Flood Happenings

•

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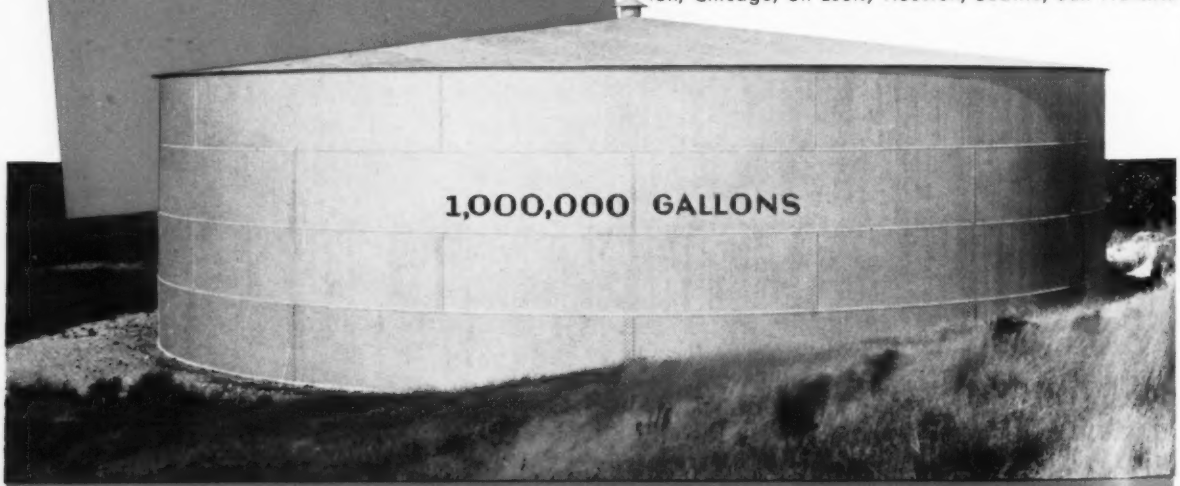

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PUBLIC WORKS and SOCIAL WELFARE

TWO items in the President's program are of special interest to the construction industry. In principle unrelated, they might in practice develop a relationship of far-reaching importance.

One of them is the proposed federal department of public works. This is not a new conception; it has been discussed for many years. In principle it offers much to commend; in practice it raises many complex questions. Its scope, the degree of its centralized control, its effect on the efficiency of existing federal construction organizations: these and other administrative aspects merit open-minded discussion by the engineering profession and the construction industry.

But aside from the administrative problems of a department designed to consolidate the normal federal works activities is the more interesting question as to how it might be related to the proposed department of social welfare. As yet we do not know exactly the scope of a welfare department but we may infer that it would handle unemployment relief. In theory such a function should mean to construction no more than it might mean to any other type of industry.

But during the last few years we have seen the federal government draft public works construction in the interest of economic recovery and social relief. Some of its recovery efforts have been soundly conceived and prosecuted. They have built useful works through normal agencies by efficient methods; they have stimulated indirect labor through private enterprise; they have respected the right of the public to receive for its funds thus invested the largest possible value. Such efforts have been helpful and constructive.

ON the other hand, federal relief administration has laid increasing emphasis on another variety of public works spending. It has rejected the organized construction industry in favor of direct employment by day labor; it has favored primitive rather than efficient methods; it has disavowed any interest in the private employment that might be stimulated by its operations; it has been more concerned in the making of jobs than in the creation of values. All this does not impugn the motive behind such efforts or indict those whose difficult task has been to carry on with them. Here it is desired merely to note one result of using public works construction to relieve general unemployment.

Having this experience fresh in mind one cannot but speculate as to whether the needs of a welfare department might not color the policies of a federal public works department. And that raises many interesting questions. For example: would a public works department confine its activities wholly to federal works? Or would it disburse federal funds for state and local works also? If the latter, to what extent might it regulate the use of the funds? Would such a department involve the establishment of a day-labor construction corps for use by federal departments? Or would it conserve the system of competitive contract? Would it be in position to require that all governmental construction practice be regulated by the needs of unemployment relief? In short, would public construction remain a productive function, directed wholly toward the efficient investment of public funds, or would it become an instrument of social welfare? And what might all this mean to the construction industry as we now know it? How would it affect the engineers, contractors, manufacturers, dealers and their employees whose jobs depend on it?

BUT the subject concerns not only those directly affected. Our total expenditures on public works—federal, state and local—normally run about $3 billion per year. The American people are entitled to have this vast investment conducted with the highest attainable efficiency. This means the maximum use of competent construction organizations on a competitive basis; it implies modern methods, materials and plant. If, in pursuance of a federal welfare policy, we would provide for those who cannot find jobs, well and good. But let us not penalize those who normally find their employment on public construction; and let us not subsidize inefficiency in the expenditure of vast public funds.

As the administration crystallizes its public works and relief programs, the general public, as well as the construction industry, will do well to see that these separate and distinct functions are kept, in practice, quite separate and distinct.

Willard Chevalier

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Table of Contents — See page 17

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FEBRUARY 11, 1937

Arch Dam of Ice Stops Slide ..... *Grant Gordon* 211

Collapse of Structure Laid to Freak Wind ..... *C. E. Garnett* 215

Drilled, Reamed Holes for Bridge Piles. ..... *J. W. Beretta* 217

Grouting Settled Backfill Under Pavement Cuts. ..... *R. A. Mayo* 218

Portland Puzzolan Cement in Bonneville Spillway ..... *R. R. Clark* 219

Flood Control Being Tested on Mississippi 222

Mississippi Flood Reports 225

Stage Construction Used for St. Louis Streets ..... *L. A. Pettus* 228

Technology and Practices in Asphalt Paving 231

New Brick Paving Practices 234

Editorials 236

Current News 238

Business Side of Construction 246

Unit Prices 248

■

Desilting Colorado River Water—With Boulder Dam forming a great stilling basin, the question has been raised as to why desilting works are necessary at the All-American Canal. Studies showing that the river will pick up a new load of silt below the dam are to be reviewed in an article to be published in the near future.

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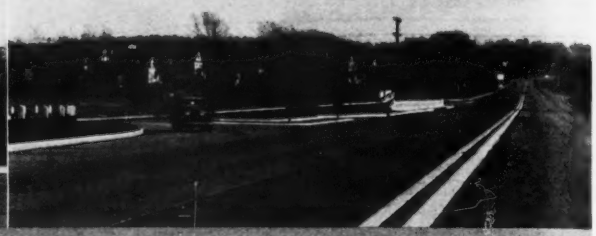

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February 11, 1937

FIG. 1—A UNIQUE DAM OF FROZEN EARTH STOPPED a slide of 200,000 cu.yds. of earth into the east excavation of Grand Coulee dam on the Columbia River.

Arch Dam of Ice Stops Slide

Basic assumptions for computing dam resistance and details of freezing procedure having record value for future operations

By Grant Gordon

Associate Engineer, U. S. Bureau of Reclamation, Coulee Dam, Wash.

THE UNIQUE RESORT *of freezing an arch of water-soaked silt across a rock gorge to check a lava-like flow of pitside earth has given spectacular interest to the excavation for a part of the rock foundation for the Grand Coulee dam. In plan the ice barrier was simple; the complexity existed in developing fundamental assumptions for strength resistance, propagation of freezing action and freezing plant capacity.* —EDITOR.

THE GRAND COULEE DAM rests on a foundation of solid granite of exceptionally fine quality directly above which is a glacial silt of ultra fine rock flour containing 20-25 per cent of colloidal material. Shear tests made on material in place have yielded a value of about 400 lb. per sq. ft. Undisturbed, and in its original horizontal bedding, the material will stand indefinitely in a vertical face of moderate height, but once disturbed, it is unstable on any slope steeper than 1 on 4 even when comparatively dry. When moistened and disturbed the material takes on the consistency of axle grease; when dry and pulverized it forms an impalpably fine dust.

Silt originally filled the canyon of the Columbia River at the dam site to a depth of about 500 ft., but the river has since cut its channel back through the silt to within 40 or 50 ft. of bedrock. During the cutting operation the river has swung from side to side in its channel, undercutting the silt slopes and removing the support formerly provided by the material in the channel section. Consequently the slopes on both sides of the river are the scenes of both ancient and recent slides. The entire mass of overburden above bedrock has been disturbed from the original bedding to a greater or lesser degree, and consequently has a low and variable degree of stability. The older slides have reached a condition of equilibrium which is stable under ordinary conditions, but the removal of the enormous quantites of overburden required for the construction of the dam has upset the conditions of equilibrium so severely and removed so much support from the toe of the ancient slides that further slides into the excavation are the inevitable result. A number of such slides have occurred, including one at the west power house, another at the west tailrace slope, one at the east pier of the highway bridge, one on the east forebay slope, and several minor slides at various points bordering the excavation areas.

Treatment of the various slides has varied with the particular conditions

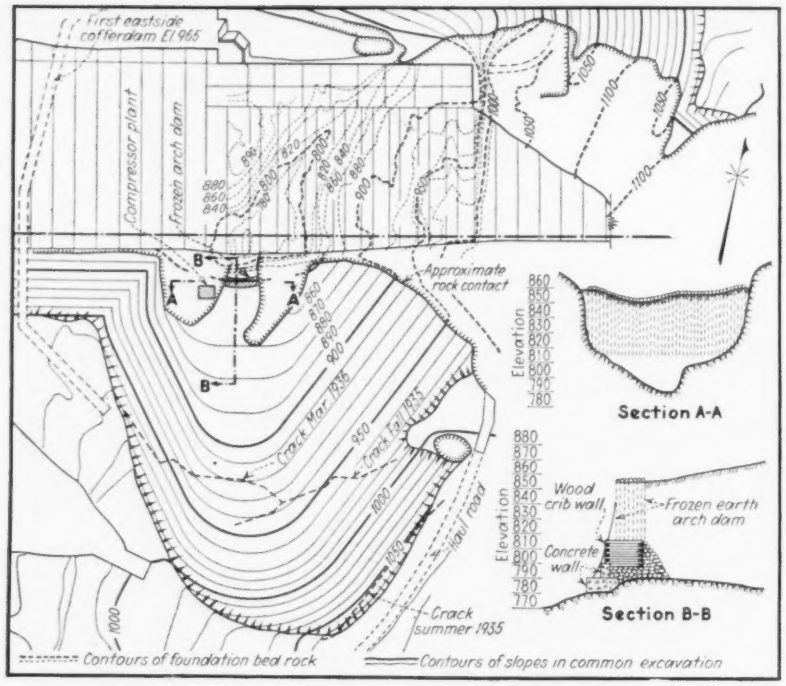


FIG. 2—THE GREAT SILT SLIDE and the rock gulch into which it flowed and stopped excavations until dammed back by an arch of ice are shown by this contour map.

obtaining in each case. Drainage by tunnels and wells has been used wherever feasible to remove underground water and prevent saturation and consequent lubrication. Surcharge has been removed in some cases, and additional weight added to the toe in others, to reduce the tendency toward sliding. A particular combination of conditions led to the adoption of a unique method in treating the east forebay slide, and to this slide attention is directed.

The critical slide

Near the center of the east excavation area, Fig. 2, the bedrock is transcepted by a long narrow gulch, 120 ft. deeper than the average elevation of the bedrock surface, which is 880, and 175 ft. lower than low water surface, which is 935 ft. The gulch runs parallel to the river and crosses the axis of the dam nearly at right angles. The walls are nearly vertical, and at a point 60 ft. upstream from the axis of the dam they are only 100 ft. apart.

From the information furnished by borings, the excavation slopes were first established at 1 on 1½ from the bottom of the gulch. But as the excavation neared the bottom, a large portion of the forebay slope gave way at the top and started to slide. To increase stability, the slopes were flattened to 1 on 2, but they were only temporarily stable, and in the early spring of 1936 the entire forebay slope was again involved in a major movement.

The slide converged upon a point just upstream from the axis of the dam opposite the south end of the deep gulch and poured through the narrows formed by its vertical walls like a flood of molten lava. The movement was so rapid that a 5 cu.yd shovel working at the toe in the bottom of the gulch could make no headway and was forced to retreat or be buried. A continuous seepage was one of the factors contributing to this slide.

During the short interval in which the bottom was exposed, the contractor constructed a small concrete arch dam across the gulch where the slide entered. The top of this arch was at El. 800. On the concrete arch the construction of a rock-filled timber crib was begun to form the toe of a rock fill across the toe of the slide area. At this period the slide was temporarily dormant, but as the crib reached El. 815 the slide suddenly became active and immediately overtopped the crib, filling the gulch and forcing the shovel to retreat as mentioned above. As this last movement precluded any concreting operations before high water of 1936, all excavation was abandoned until after high water, and the entire east excavation was flooded.

During high water the slopes above water level were excavated to slopes varying fromm 1 on 2½ to 1 on 3.13 opposite the south end of the gulch, to remove additional surcharge from the slide. As soon as unwatering operations were begun after high water, a break appeared at the top of the slope and the old slide started to move toward the gulch. The pool was lowered from El. 955, the top of the east cofferdam, to El. 850. The slide followed it down. At this point pumping was reduced to that required to hold the water surface stationary. At the time when operations were started on the freezing, the slide was moving into the pool at a velocity of about 2 ft. an hour.

Ice dam projected

The slide now presented a real problem. To flatten the slopes until they were stable would be an enormously expensive job; to attempt excavation at the bottom of the gulch without some stabilization would invite disaster. Since drainage alone could not be relied upon and no economically feasible method offered any hope of success, the expedient of freezing an arch across the slide area between the steep rock walls of the gulch was decided upon. The top of the arch was taken as El. 855, and, to promote freezing by maintaining a saturated condition,

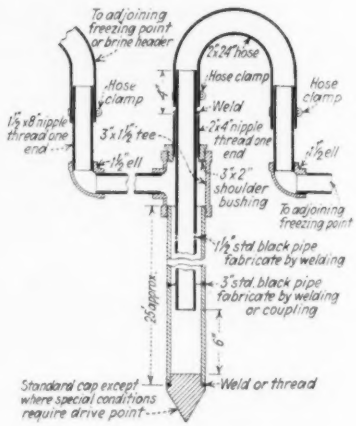


FIG. 3—FREEZING POINTS numbering 377 were sunk deep into the slide on the location of the ice dam and cold brine was pumped through them to freeze the water soaked silt.

and to maintain sliding stability by its weight against the front of the arch, the pool level was held at El. 850 until the operation was completed. The operations were timed so that freezing could begin as soon as the surcharge back of the arch was removed and be completed before the contractor was ready to renew excavation operations in the floor of the gulch.

The arch was intended as a temporary measure only. It was designed to restrain the sliding material for such time as was required to remove the desired excavation and to concrete a sufficient height of dam to get out of danger from further slides. The cost of the arch, as roughly estimated, would be covered by the saving in excavation.

besides which there would be saving in time and money to the contractor.

Once the course of action was decided upon much had to be done. No criteria existed from former frozen arches. The only information available about similar operations was meager and offered little that could be used. The freezing method invented by F. H. Poetsch of Prussia, used in sinking deep shafts, was the nearest approach to anything similar, and available information on operation and costs offered little basis for comparison. Accordingly design and cost data for the entire setup had to be worked up from scratch. Initial assumptions were made from known refrigeration data, the time available for completion of the arch ascertained, tentative designs sketched out and the Pacific Northwest was scoured for available refrigerating equipment which might be rented or purchased. Finally two used ammonia compressors were located which had a combined capacity of 80 tons of ice per day and were available at once. Using this capacity for one known factor and the length of time available for another, the refrigerating plant was designed, the arch laid out, and the number and type of freezing points determined.

Design of arch

The sliding material completely filled the gulch between its vertical rock walls to El. 855, at a point above the concrete arch and the rock-filled timber crib resting on it. The top of the timber crib was at El. 815. An arch was projected which would derive its foundation support from the timber crib and spring against the rock walls of the gulch. A radius of 105 ft. was chosen to fit the timber crib and abutment walls; a shorter radius would have decreased the thickness required and consequently the volume to be frozen, but the arch would have been unsupported in the center section and a tendency toward lifting was feared. Furthermore the arch of shorter radius was more inclined to slip along the abutments.

Assuming a strength of 200 lb. per sq.in. for the frozen material, and taking the weight of the sliding material as 90 lb. per cu.ft., with 75 per cent of full liquid pressure against the arch, a thickness of 10 ft. was required at the base, as computed by the cylindrical formula. To provide a suitable safety factor the final arch thickness chosen was 20 ft.

Assuming that three weeks were available for freezing operations a spacing of 30 in. between freezing points was used. To arrive at this figure it was assumed that with 10 deg. brine 6 in. of material would be frozen in two days, 12 in. in eight days, and 15 in. in 20 days. The figures were based on the rate of freezing of ice and checked by considering the insulating effect of ice around the points The figures were assumed to be conservative because the specific heat of the material to be frozen was only .45 above the freezing point and only .31 below. The material in the arch had an average water content of 32 per cent by dry weight. Actually the material behaved somewhat differently than anticipated because of its exceptionally high rate of heat conductivity. This resulted in the whole mass lowering its temperature at a surprisingly uniform rate, a distinct advantage in the freezing of the arch.

FIG. 4—A CLOSE UP VIEW showing the freezing points in the top of the frozen earth dam which stopped a slide of 200,000 cu.yd. of earth into the excavation on the east side of the Columbia River at Grand Coulee dam.

The freezing plant

An ammonia-brine refrigerating system was selected as most satisfactory. The plant consisted of three separate systems: (1) the brine system which cooled the material to be frozen, (2) the ammonia system which cooled the brine, and (3) the cooling water system which cooled the ammonia. The brine was cooled by the expansion of liquid ammonia in two brine coolers and was pumped by a 7½ hp. centrifugal pump through the coolers and down the side of the gulch through a 3-in. riser to the top of the arch, where it was divided through two 3-in. supply headers.

The freezing points were constructed as shown by Fig. 3. The brine was forced down the 1½-in. pipe, returned through the annular space between the 1½-in. and 3-in. pipes, and emerged through the tee. The outlet end of one point was connected to the inlet end of the next in series by means of a 2-in. three-ply rubber hose, fastened by two metal clamps on the ends. The points were 43 ft. long on the average and extended from ground surface to the rock-filled timber crib; sixteen points were connected in a group, in series between headers, and the flow was regulated through the series by valves on both headers. The points were spaced 30 in. center to center both ways. The arch was formed by eight parallel rows of points plus about 25 extra points on each abutment haunch, to increase the bonded area and lessen the tendency toward sliding. A total of 377 points was installed and in service.

The freezing points were installed in the following manner: The 3-in. pipes were driven to place by two rigs; one was a caterpiller-mounted, walking-beam type churn-drilling outfit, and the other was a gasoline-powered diamond drill rig mounted on a derrick barge. The barge floated on the pool in the excavation, which stood at El. 850 during the entire operation. The 3-in. pipe was furnished in stock lengths of 21 ft. plus, and most of the points required two lengths. The first length was driven by a 300 lb. drop-hammer striking against a heavy drive coupling; the second was coupled on and the point driven until it came up on the timber crib below. Each rig averaged about one point completed per hour. The barge rig drove the points at the edge of the pool and as far inland as it could reach, while the well rig drove the remainder. It was found that the blunt end formed by the standard pipe cap allowed the points to be driven more nearly plumb than a sharp point, because of the tendency of the latter to deflect when gravel or other hard object was struck. The type of cap used on the pipe is shown in the accompanying drawing.

At the beginning both machines were operated at full capacity; after 6 weeks either machine maintained the low temperature of the arch handily. Under full operation the temperature difference maintained between incoming and outgoing brine was 8 deg. F. As the arch temperatures dropped, this difference remained constant until an arch temperature of 30 deg. F. was reached, or until most of the latent heat of fusion was removed and the moisture was frozen. Below this temperature the difference became less as the refrigeration requirement lessened. The 8 deg. difference corresponded to a net effective refrigerating capacity of 70 tons of ice per day.

The brine circulation velocity was controlled by valves to balance the refrigeration between the 24 groups of points, and was maintained at about 1 ft. per second through the freezing points. The total circulation required was 170 g.p.m. Because of the temporary nature of the set-up, ordinary rock salt was chosen for the brine because of its economy. The solution was maintained at about 95 per cent saturation, or a relative density of about 1.2. The brine had a freezing point of about —4 deg. F. when still, but was kept in constant motion. About 20 tons of salt were required to charge the system; it was delivered to the job in 120 lb. bags.

Under full load the compressors were operated with a back pressure of 125 lb. per sq.in. and a suction pressure of 20 lb. per sq.in. As the required capacity dropped off, the suction pressure was held at 10 lb. per sq.in. and the back pressure dropped to 105 lb. per sq.in. A total of 2200 lb. of ammonia was required to operate the system efficiently, very little being lost by leakage. The ammonia was admitted to the system through a valve just ahead of the expansion valve.

The condensing water from the river cooled the compressed ammonia to the condensation point with a 6 deg. F. temperature rise. The temperature of the river water varied from 53-60 deg. F. Since ample quantities of water were easily available there was no point in being economical with it and the pump was operated at normal capacity.

Difficulties reviewed

When driving was begun, the material in the center of the arch was moving downstream at a velocity of 2 ft. per hour. During the operation, however, the material reached a fairly stable state of equilibrium, and for the week prior to the start of freezing no movement was detected. As soon as freezing began, the arch began to develop load. It is believed, and analysis bears out the conclusion, that the major portion of the load, and consequently the movements in the arch, were due to expansion of freezing material. The quarter points of the arch moved downstream at a greater rate than the center, which is contrary to the normal behavior of a uniformly loaded circular arch. Since all the material composing the arch had been violently disturbed and broken up during sliding, there was a considerable volume of voids to be consolidated. The major movement was downstream at a steadily decreasing rate, but daily observations show an oscillation upstream and down as the material adjusted itself to its self-imposed load. No movement was observed on the abutments. The observations were taken on the ground surface, and it is believed that the lower 30 ft. of the arch did not move to any great extent.

For a description of Grand Coulee dam, the construction plan and the extensive belt-conveyor earth-handling operations the reader is referred to the special Grand Coulee Dam issue published by Engineering News-Record, Aug. 1, 1935.

The arch was designed to derive its main support from the timber crib beneath, and it was also assumed that drainage of ground water from the slide area would take place through the crib. However, it was believed necessary, during the period before the earth in front of the dam was removed, to sink a drainage shaft 30 ft. upstream of the arch to El. 800 to remove ground water and thus prevent development of liquid pressure against the drozen arch. This well produced a flow of about 25 gal. per minute. As soon as excavation in front of the dam was completed down to crib level, drainage took place as anticipated, and the well was no longer required.

No particular difficulties were experienced in maintaining a balanced rate of freezing in the various groups of points by a simple manipulation of valves. Thermometer wells made of ½-in. pipes 10 ft. long were driven at frequent intervals throughout the arch to give an accurate check on the rate of freezing.

Some difficulty was experienced with air trapped at the high points of the hose connecting adjacent freezing points, until the expedient was adopted of making a small hole in the top of each hose, allowing the air to escape, and then plugging the hole with a toothpick or shingle nail.

Considerable difficulty was experienced in driving the second joint of pipe because of the tendency of the driving coupling to batter the threads on the pipe. It was found that a heavy hammer, 300 lb. in this case, drove the pipe easier and with less damage to the threads than a lighter hammer. All the leaks in the freezing points were at the couplings joining the lengths of pipe. Great care was necessary in joining the two lengths to make them withstand driving.

A total of 10 points failed while in service; these invariably broke at the coupling. Thrusts developed by expansion during freezing bent the pipe out of line as much as a foot in some cases before failure took place. The upper portion of the arch, being less confined, moved more than the lower portion. This put a shearing stress, and in the case of the vertical component, a tensile stress on the coupling. Points which failed were removed from service, and no difficulty was experienced in maintaining low temperatures without them because of the high rate of heat conductivity of the material. Blasting operations on the rock excavation for the foundation of the dam complicated matters somewhat. Flying rocks cut off the hose connecting the adjacent freezing points, and jars from the blasts frequently broke points already stressed to the limit. It was necessary to be constantly on the alert to avoid heavy losses of brine from leaks through broken hoses and points. To minimize these losses, a float switch was installed in the brine surge tank so that a drop of .01 ft. was sufficient to ring a bell to warn the plant operator. The scheme worked satisfactorily. Brine which leaked out through broken points found its way down the points and drained away through the timber crib without causing any particular damage.

Cost $30,000

The arch as constructed and the operation of the plant for two months cost as follows:

Equipment for installation and operation of plant for 2 months, including rentals	$6,000
Materials and equipment purchased	14,000
Labor—installation and operation	10,000
Total cost	$30,000

Interesting unit costs include the following:

Salt	ton	$11.10
Anhydrous ammonia	lb.	.165
Hose—rubber, 2 in. dia.	3 ply ft.	.44
Hose clamps	each	.12
3 in. std. blk. pipe	ft.	.32
1½ in. std. blk. pipe	ft.	.115
Sawdust—loose measure	cu.yd.	.70

The total cost of $30,000 represents the freezing of an arch 100 ft. long, 20 ft. thick, and 40 ft. high, containing about 3000 cu.yd., a unit cost of $10 per cu.yd.

Against the cost of the frozen arch is credited an actual saving of not less than 30,000 cu.yd. of excavation otherwise required at $1 per cu.yd., the contract price. In addition a saving of several weeks of the contractor's time in excavating and concreting the bot-

tom of the gulch, with its attendant speeding up of all operations contingent upon it and consequent saving of overhead and contingent expense is to be credited. And also to its credit is the stabilizing of the entire forebay slope which converges upon the south end of the gulch. The amount of yardage which might be involved is impossible to estimate, but to anyone who has observed the movements of that slide over its period of activity, the 30,000 cubic yards actually credited seems but a small fraction of what would certainly slide if a shovel ever started excavating where the frozen arch stands.

Collapse of Garage Structure Laid to Freak Wind Gust

Opinions agree that flat roof steel structure without walls was victim of unusual loading—Design met building code requirements

By C. E. Garnett

Architect and Civil Engineer
Daytona Beach, Fla.

THE COLLAPSE of the roof of a parking garage building in Daytona Beach, Fla., during a storm last summer has been the subject of considerable conjecture and study since. The consensus of opinion, six months after the accident, apparently is that the primary cause was a sudden gust of wind, that the design and workmanship met accepted standards, and that the first publicized theory of the cause of the collapse, namely, rain accumulation on the roof, is untenable because the roof, although flat, has no parapets and only a 2-in. gravel stop along the edges.

The structure that collapsed was not a building in the true sense but merely a truss-supported roof 100 ft. wide and 111 ft. long built as an extension of an existing service station which occupies the front part of the lot. The 111-ft. length was divided into five 22 ft. bays. The 100-ft. wide bents consisted of trusses 4 ft. 10 in. deep resting on three columns, those on the centerline being 8-in. 31-lb. H-sections while those at the ends were slightly lighter. Knee braces were used between the columns and trusses. Purlins were 9-in. channels attached to each truss by two ¾ in. rivets. Longitudinal struts as well as rod x-bracing was used between the trusses. The walls were open to the weather except for a metal apron descending 6 ft. from the roof and a base wall of concrete block 4 ft. 9 in. high. The roof was of 2x6 T & G timber sheathing nailed to 2x4's bolted to the purlins.

The older building on the front of the lot, with somewhat similar framing, is shown in Fig. 2. The driveways either side of the showroom and parts building were used for service and for car storage. The lack of enclosure walls on the north side of the old building and on the sides and rear of the new building and the presence of these driveways closed in by adjacent buildings created a condition favoring drafts and cross currents.

It is the writer's opinion that wind currents entering through the driveways and through the south wall near the back of the building met at a common point south of the center column under truss No. 4 (Fig. 3) and, being deflected upward, raised the roof at this point and then allowed it to drop suddenly. The counter currents gave the wind a gyratory motion so that it next struck the opposite side of the roof under truss 3, twisting the center columns at their weakest points, the knee brace connections. After the collapse began, it was inevitable that the rest of the roof must follow. The twisted state of these center columns is shown in Fig. 1. The column under truss 1 rotated about 100 deg. clockwise, under trusses 2 and 3, 170 deg. counter-clockwise, and under truss 4, 180 deg. counter-clockwise. That there was considerable wind of a gusty nature is evident from the fact that rain was driven some 90 ft. inward from the front of the building.

Harry M. Griffin, Daytona Beach, was the architect of the building, and the structure was designed by W. R. Stults, his office engineer. Mr. Griffin attributes the cause of the collapse to a "freakish windstorm." Mr. Stults states that the structure "was designed to support a live load equal to 4 in. of water over the entire area; the eleva-

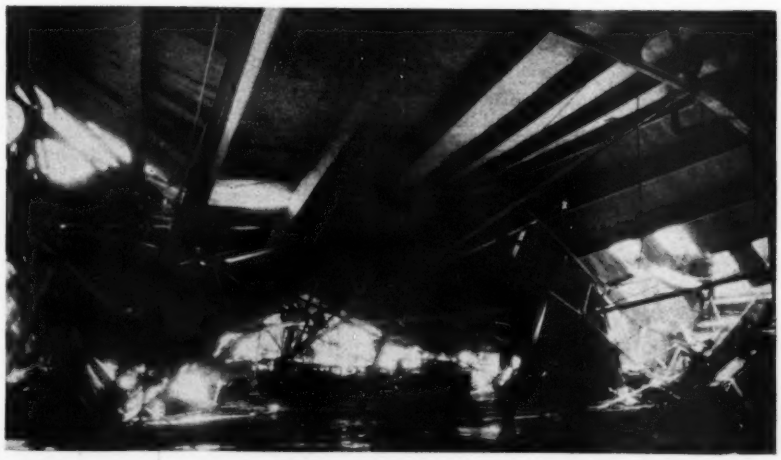


FIG. 1—WRECKED FRAMEWORK of car parking shelter in Daytona Beach, Fla. Comparing with Fig. 3, view is taken looking northwest. Truss 1 is in upper right corner.

FIG. 2—ORIGINAL BUILDING on front of lot through whose driveway wind currents could reach the interior of the rearward extension. The framework at the right is typical of that of the failed portion.

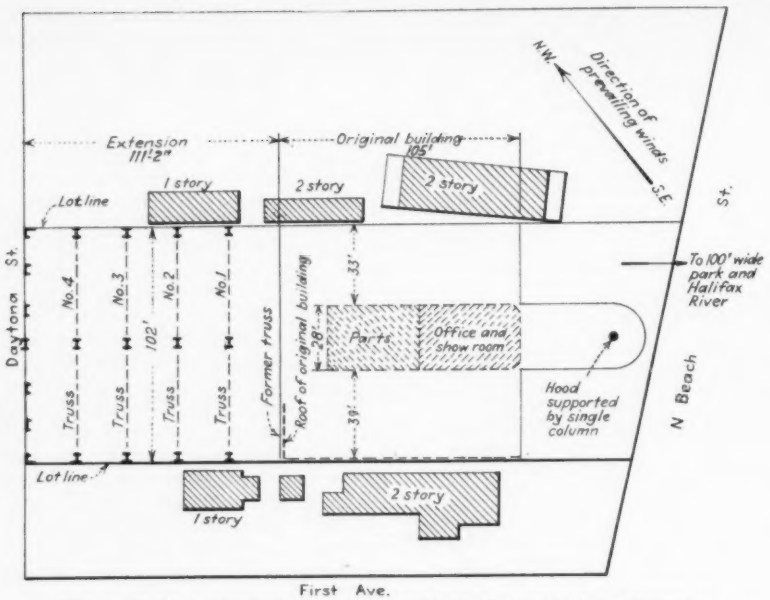


FIG. 3—PLAN of service station and car parking shelter and adjacent structures.

tion at the edge of the roof was not sufficient to retain this water load. A careful re-check of the truss design shows that about 1 ft. of water over the entire roof would be required for failure. The state of the wreckage in general indicates, he believes, "that the building suffered a suddenly applied load in the nature of a blow delivered practically simultaneously over nearly the entire area. The third column from the west on the north side gives every evidence of having first been lifted, as the foundation bolts are pulled in two and the column base clips bend downward. Had such a lifting occurred over the entire area, even though slight and followed by a drop, it would have served to have doubled the effect of both the water load and the dead load and would be sufficient to have caused failure." In the design the total vertical load was taken at 40 lb. per sq.ft. half of which was live load. No wind load was considered in the design, which was made for a closed wall building which the owner later demanded to be left open.

Two other engineers were invited to report on the accident. N. W. Green, consulting engineer, Jacksonville, Fla., found "the design and details to be in accordance with standard practice and all structural members to be adequate for stresses encountered in buildings of this type. It is my opinion that failure of the roof framing was the result of a sudden and unusually violent vertical wind stress of very limited area and of short duration. From the location and position of various members of the distorted roof framing, it is my belief that an instantaneous combination of exterior compression with a partial interior vacuum produced an impact on the structure both unusual and far in excess of the stresses usually provided for. I have no hesitancy in recommending that the building be rebuilt according to original plans and specifications."

No variations between the shop drawings, which were made in accordance with the general design, and the actual structure were found by J. Stoltenborg, consulting engineer, Jacksonville, who also reported the workmanship to have been very good. Mr. Stoltenborg concluded that "inasmuch as an accumulation of enough water on the roof to cause an overloaded condition was impossible due to the fact that there were no parapet walls but only a 2½ in. gravel stop at the eaves, that failure must have been caused by some freakish wind condition, probably very small in area and of high intensity at the center."

Airport Support for Municipalities

AMERICAN cities have invested large sums to provide airport facilities for both public and private plane users during the past few years but benefits other than self-advertising and goodwill have flowed to state and federal as well as city governments. The time has come when definite and complete federal action is needed for a national system of civil airways and airports, in order that the cities may know where they stand for the future.

This is the gist of a report made public, January 13, by the American Municipal Association. The report includes a resolution adopted at a recent meeting of the association's executive committee in Washington, D. C., which calls attention to the problem and recommends a complete projected national system of civil airways, including the establishment of financial responsibilities of the various agencies benefiting by such airports.

At present, the report points out, about 2,000 airports and landing facilities are operated in the continental United States, of which 475 come under the classification of municipal airports. About 200 of these 475 airports, representing an investment of several hundred millions of dollars, are utilized as terminal facilities for the national civil airways system that was set up in July, 1936. In most cases the report finds the major part of these investments has been made at the expense of local taxpayers, and without consideration of the broad public service or national service rendered by these facilities. Figures on 36 airports show a total capital outlay of $34,198,614 and a total annual expense of $1,550,087.

There are three functions of an airport serving as a part of an established airway system, the report finds: First, local use; second, service in interstate commerce to a population in an area ranging from 50 to 400 miles in radius; third, use for national defense purposes. Rapid growth of air transportation is fast leaving most municipal terminal facilities in an obsolete condition, it is pointed out, and to keep step with the development in air transport equipment, immediate further planning and development of municipal air terminal facilities becomes necessary.

Federal agencies, in the opinion of the committee, should consider federal responsibility for any projected civil airways program on a basis similar to the federal responsibility in the national highway system and rivers and harbors. Such consideration, however, would be for airports in accordance with their federal value—as stops on the national airways or for national defense—after proper coordination through state aeronautical officials and in compliance with federal requirements governing design and construction features, maintenance and operation of the civil airways terminals.

Groundwater Data Compiled

INFORMATION obtained on water levels and artesian pressures in observation wells located in 25 states in the territory of Hawaii for the year 1935 has been compiled by the U. S. Geological Survey and published as Water Supply Paper No. 777. A similar report on groundwater levels in 1936 is now in preparation.

Drilled, Reamed Holes For Bridge Foundation Piles

Cast-in-place piling, adapted from building practice, aids in providing an economical concrete bridge of continuous slab type

By J. W. Beretta
Consulting Engineer, San Antonio, Texas

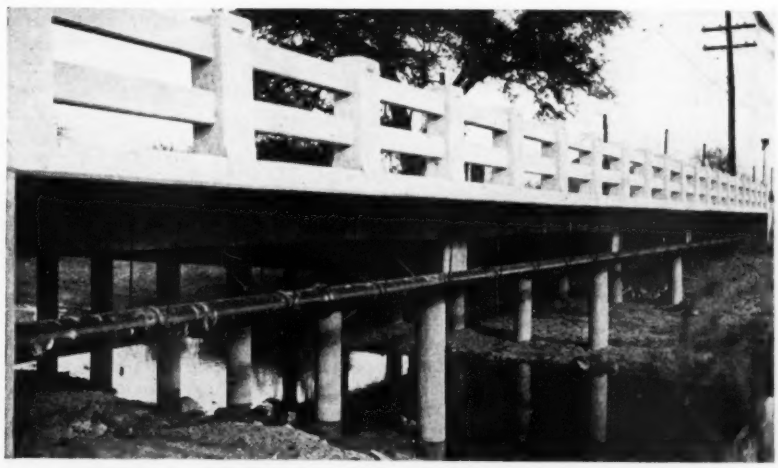
THE NEW Mitchell St. bridge across San Pedro Creek in San Antonio, Texas, is characterized by several departures from conventional reinforced concrete bridge construction practice, principal among which is a simplified foundation design that permitted handling subaqueous excavation with less difficulty than is often experienced under dry foundation conditions. Of continuous solid slab type on a skew of 16 deg., the bridge is carried on five supports and has two 36-ft. center spans and two 34 ft. end spans. The roadway is 36 ft. wide flanked by two cantilever sidewalks each 7 ft. wide. The roadway slab is designed for standard H-15 live load with specified impact factors, and is of the solid, haunched type. Intermediate supports consist of drilled, reamed, cast-in-place pile bents topped with cross girders cast monolithic with the roadway slab. The end supporting abutments are of the buttressed type, likewise supported on drilled, reamed, cast-in-place piles. Heavy railings of reinforced concrete are used. The wearing surface of the roadway is monolithic concrete topping with standard highway type finish.

Although the use of drilled, reamed, cast-in-places piles has been extensive in the San Antonio area for the foundations of buildings, their first application to a bridge is marked in this structure. At the bridge site a uniform and level strata of compact clay was encountered at about 6 ft. below average water level in the creek. Over this was a variable thickness of sand, silt and gravel permeated with water.

Holes for the 18-in. circular piles were excavated by a modified gasoline rotary well drilling rig mounted on an old truck chassis, a type of rig widely used to drill shallow water wells. In practice, the holes were first drilled 20 in. in diameter through the sand, silt, and gravel and about 1 ft. into the clay strata. Into this hole was inserted a piece of steel pipe casing with ¼-in. walls and welded seams, which was then driven several inches into the clay to seal off the water. Following this operation the water was bailed from the hole, the space around the outside of the casing filled with fine silica sand, and the drilling continued with an 18-in. auger to a depth of 7 to 8 ft. into the clay strata. Using a reamer with folding blade, a working chamber or bell 3 ft. in diameter at the bottom and with conically sloping sides was next excavated. A laborer equipped with short handled pick and shovel then entered the hole and expanded the bell to 5 ft. in diameter at the bottom with conical sides sloping upward at 60 deg.

Into this finished shaft was inserted the vertical steel reinforcing bars with spiral reinforcing, and the entire bell and shaft were filled with concrete to the top of the casing. At this juncture

FIG. 1—SUBMERGENCE during floods may subject the new Mitchell Street bridge in San Antonio to large lateral forces. To resist them, piling, bent caps, continuous deck slab and railing are built monolithic.



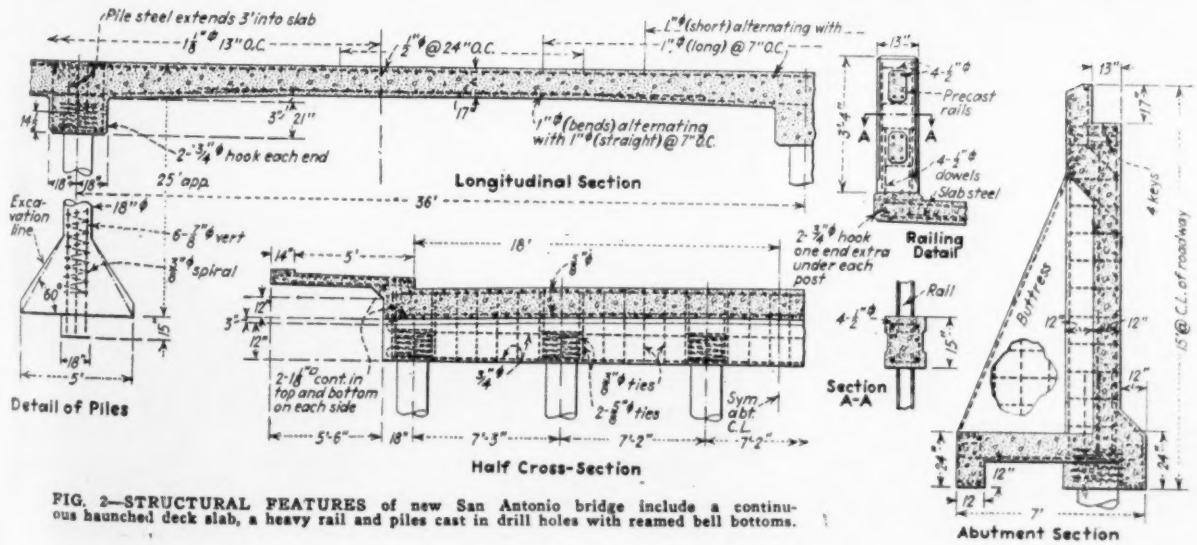


FIG. 2—STRUCTURAL FEATURES of new San Antonio bridge include a continuous haunched deck slab, a heavy rail and piles cast in drill holes with reamed bell bottoms.

the casing was withdrawn allowing the plastic concrete to slump into contact with the sides of the shaft. Collapsible metal forms were then used to continue the pour of the pile shaft above ground to its proper elevation for supporting the bridge. In the case of the abutments the piles do not extend above the ground level. Also in the one pile bent which was placed in running water, it was necessary to leave the subsurface casings in place permanently

As there is a possibility of the complete submergence of the bridge during times of excessive flood, the slenderness achieved through continuous slab design adds greatly to the practicability of the bridge, aside from a saving in cost. Such submergence requires maximum lateral stiffness which is obtained by restraining the heads of the piles, by providing bearing plate details at the abutments designed to resist lateral stresses and by keyways to resist uplift. The ends of the piles extend about 14 in. into the cross girder, with spiral hooping around each pile head as a part of the cross girder reinforcing. In addition, the vertical reinforcing of the pile extends through the cross girder to the level of the top reinforcing in the roadway slab where it is bent into a horizontal position, assuring a rigid connection.

Expansion joints

Expansion joints are provided at each end of the bridge on the abutments. These joints are of a type new to this section of the country and consist of a poured rubber latex and cork compound placed in accordance with the manufacturer's directions. The expansion joint material was placed after the removal of forms and seemed to bond well with the concrete surfaces of the joint. Drainage of the roadway and sidewalks is provided by slight gradient from end to end of the bridge. Catchbasins are provided on each side of the street behind the west abutment; drains extend through the walls.

Through the simplification of design, details and construction practice an unusual economy was realized in this bridge. Truck-mixed concrete was used throughout, laboratory controlled to 3,000 lb. per sq. in. A cement of high early strength characteristics speeded up construction time by enabling early form stripping and shore removal. The total cost of the structure, including engineering, railings, and electrical work, was $19,747.50 or about $2.82 per sq. ft. of bridge. The bridge was designed and supervised by the J. W. Beretta Engineers, Inc., San Antonio, Texas, for the city of San Antonio under the direction of I. Ewig, city engineer. The general contractor was Chamberlain & Strain, and the subcontractor for the foundations A. H. Beck Foundation Co., both of San Antonio.

Grouting Settled Backfill Under Pavement Cuts

Breakdown of pavement patches over backfilled cuts reduced by sand-cement grout injection

By Robert A. Mayo

Construction Engineer, Ransome Concrete Machinery Co., Dunellen, N. J.

IT HAS ALWAYS BEEN a difficult task to restore backfill under pavement cuts so that it would not settle away from the replaced slab. In recent years the problem has been intensified for gas companies because they have had to make many more cuts due to the necessity of furnishing a dryer gas for automatic gas regulating mechanism in refrigerators and water heaters. Removing the tars and oils has caused the yarn caulking of the pipe-line joints to shrink and leak and the old joints have had to be replaced by iron collars with external gaskets. Each replacement called for a 4 ft. square pavement cut and backfill and sometimes a cut had to be made for every 12 ft. length of gas pipe. Backfill shrinkage thus became a major problem.

Faced with this problem, Frank A. Engel and Dennis J. Manning, superintendent and assistant superintendent of distribution for the Elizabethtown Consolidated Gas Co., Elizabeth, N. J., decided to solve it by the method which had been used for years to consolidate ground and to fill fissures under dams. A pneumatic grout mixer and placer was purchased and the cavities under several patches were grouted; this proved so simple and so effectual in stopping settlement of the patch that after more experimentation the following procedure, which is now standard with this company, was evolved.

The grouting equipment consists of a light truck on which is mounted a 110-cu.ft. gasoline-driven compressor which furnishes air for a jackhammer drill and the grout machine. The grouter is carried on a two-wheel frame which is trailed behind this truck and another truck follows, carrying sand and cement. Ninety days after the street has been patched, or 120 days if the weather has been very dry, the grouting crew passes over a series of patches. A hole $1\frac{7}{8}$ in. in diameter is drilled in the center of each patch, these patches being in most cases 4x4 ft. The foreman then drops an electric light, the power for which is furnished from a storage battery, into each hole and inspects it to see if grouting is required. If the apparent settlement is $\frac{1}{2}$ in. or less, no grouting is specified; generally about one-third of the patches will require no grout.

The grouter is a cylindrical tank with a conical bottom leading to a nozzle and it is mounted in the trailer frame in such a manner that the nozzle can be lowered into a hole. There is a swivel plate and rubber gasket around the nozzle; thus the weight of the machine is used to keep the gasket tight against the surface of the street. The grouter has a 3-cu.ft. tank but is gen-

GROUTING OUTFIT of compressor truck and a mixer on a two-wheel trailer restores bearing of pavement patches over settled backfill.

erally used with 2-cu. ft. batches. Water is put in the machine and cement and sand slowly added; mixing is done rapidly and thoroughly by agitation. The charging door is then closed and air pressure applied to force the grout through the nozzle. When the cavity is filled, two men roll the grouter to the next hole and prepare another batch; 2 cu.ft. batches have been found about correct for completing most patches.

The grout is a rather soupy mix of one part cement to six parts of sand; fifteen pounds pressure is generally used for forcing in the grout. Mr. Manning has installed a pressure regulating valve between the compressor and the grouter which can be adjusted to any desired pressure; this prevents a careless operator from using too much pressure which might raise the patch, or waste grout by forcing it under the adjacent slab or down some channel to the sewer.

An unusual example of the efficiency and economy of this system occurred on one of the principal streets of Elizabeth recently. Fifteen years ago, a 20-in. main had been recaulked; there were 30 patches originally each 5x5 ft. One patch settled and had to be removed and an area 7x7 ft. replaced. An inspection of the remaining patches showed that the backfill had settled away from the slab an average of 4 in. It was estimated that to barricade the street, break out and remove the old patches, add more backfill, and then replace the patches would cost about $35 each or about $1000 for the job. Three men grouted the remaining 29 patches in half a day's time, and without seriously interfering with traffic.

Where large-size mains pass under wide and expensive pavements, it is often much cheaper to tunnel than it is to cut the pavement, but most street and highway departments have forbidden this because it is impossible to ram the backfill properly so that eventually the edges of the tunnel will not settle, permitting the unsupported pavement to crack and break. Mr. Manning has satisfactorily solved this difficulty by drilling holes, 8 ft. apart, over the center line of the tunnel. Through these holes, grout is introduced to compact the backfill and reinforce the pavement.

Portland-Puzzolan Cement in Bonneville Spillway Dam

Comparison of cements by heat of hydration, water gain, and permeability favored puzzolan, which was specified as to content and manufacturing process. Observations on first 275,000 cu.yd. placed confirm expectations

By R. R. Clark

Designing Engineer
U. S. Engineer Department
Portland, Oregon

PLANNING CONSTRUCTION of the main spillway dam across the Columbia River on the Bonneville project required that a rather unusual set of conditions be met. Selection of the most suitable type of cement was accepted as one of the problems because it was believed that best results would be obtained if the cement had certain qualities that fitted the particular needs in this case. However, decision on this point was not reached until a comprehensive series of tests had been made in the cement laboratory at the University of California under the direction of Prof. R. E. Davis.

The spillway dam on this project differs from other large concrete dams recently built in that the ratio of height to width of base is relatively low (*ENR*, Nov. 1, 1934, p. 547). Vertical lift gates extend the entire length. This results in making the mass section trapezoidal in shape between piers and divides it into blocks at 60-ft. intervals by contraction joints to be grouted. The width of base is approximately 135 ft., and there is no provision for contraction. This feature, without other design consideration, might induce objectionable vertical cracks parallel to the axis of the dam as the result of temperature changes.

The normal compressive stresses will be low; hence high-strength concrete is not of primary importance. Again, since the dam will seldom be exposed to drying, and then only for short intervals, a low contraction coefficient of concrete on drying was not a necessary requirement. The construction season at Bonneville is between summer floods; concrete therefore will be placed during winter months, September to April, when placing temperatures are from 40 to 60 deg. Mass concrete generally is placed in lifts 5 ft. thick with the surface sloping downward in an upstream direction to facilitate the cleanup on each lift. An interval of three days elapses before a lift is covered.

In view of these conditions, it appeared that a cement of low ultimate heat generation should be employed, but that even under the prevailing conditions of low placement temperature a large percentage of the heat of hydration should be liberated in the early ages so that as much of this heat as possible might be dissipated from one lift of concrete before the next lift was placed. It also appeared that the cement should be of a type that would produce a plastic and workable concrete in mixes containing one barrel or less of cement per cubic yard.

Three types of cement

In the endeavor to find the cement most adaptable to these conditions, three types were considered, as follows:

Type *A*.—Modified portland cement, moderately low in tricalcium aluminate, but otherwise of normal composition, similar to that used in Morris Dam.

Type *B*.—Low-heat portland cement, low in tricalcium aluminate and tricalcium silicate, similar to that used in Morris Dam and Boulder Dam.

Type *C*.—Portland-puzzolan cement, containing portland cement interground with a siliceous material capable of reacting with lime liberated by the portland cement, similar to that used extensively abroad.

To study adequately the relative merits of the three types of cement prior to the selection of one of them, an extensive series of investigations was undertaken at the University of California for the War Department. The results of these tests are partially summarized in the following.

Table I gives the results of tests for water gain and permeability with 0.8 bbl. and 1.00 bbl. of cement per cu.yd.,

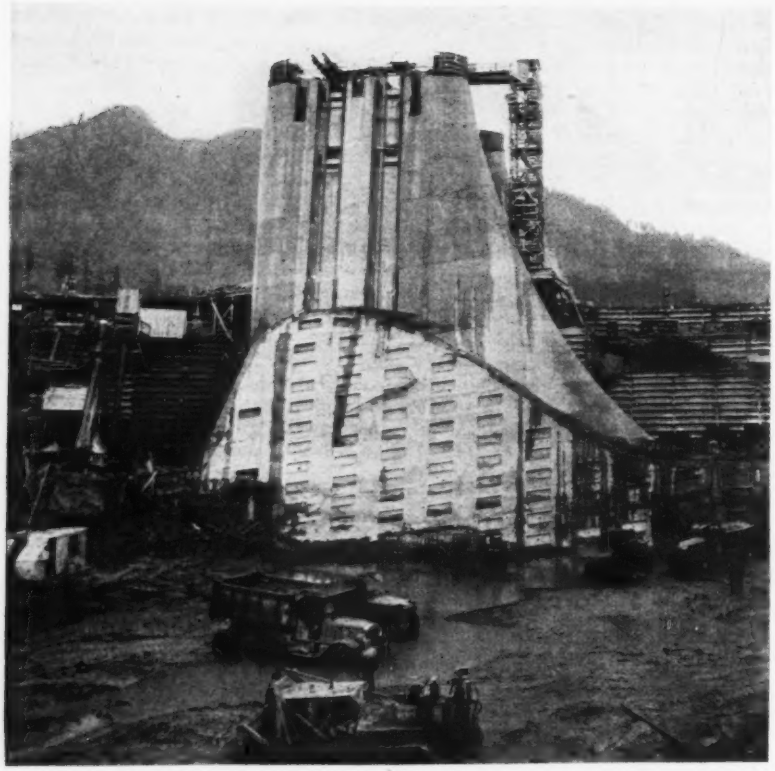


FIG. 1—TYPICAL PIER of Bonneville spillway dam built of concrete made with portland-puzzolan cement.

which were averaged to give a value of 0.9 bbl. per cu.yd. The table indicates that the water gain of a *C* cement of 1800 fineness is equal to that of an *A* cement of 2200 fineness and is one-half that of a *B* cement of 1800 fineness. Similarly the permeability factors are superior for the *C* cement in so far as these properties are concerned. Numerous other tests indicated that both water gain and permeability are reduced by increasing the fineness of the cement or by increasing the richness of the mix. These results strengthen the assumption that low water gain and low permeability go together.

Table II gives the results of heat of hydration studies in which heat of hydration is recorded in calories per gram with water/cement ratio 0.40 by weight; control temperature 70 deg. F. Under comparable conditions the heat of hydration of the portland-puzzolan cement was found to be higher than that of a low heat portland cement and substantially lower than that of modified portland cement.

In Table III the cement factor is 1 bbl. per cu.yd. of concrete adiabatically cured with an initial temperature of 40 deg. F. Values are for an average specific surface of 2,000 sq.cm. per gram.

Plastic flow as measured by the sustained modulus of elasticity (unit stress divided by the sum of the elastic and plastic unit deformations of concrete made from the *C* cement was higher than that of the *B* cement, but lower than the *A* cement. At later ages the sustained modulus of the *C* cement was lower than for either of the other two cements.

Portland-puzzolan specifications

As the results from these tests were made available, it became evident that portland-puzzolan cement was best adapted to the special requirements at Bonneville and the decision was made to use it. This decision was based on the following properties:

1. Improved workability.
2. Freedom from segregation and water gain.
3. High factor of impermeability as compared with low heat portland cement.
4. Greater percentage of total heat of hydration developed at early ages and less total heat of hydration in comparison with the modified portland cement.
5. Relatively high early tensile and compressive strength coupled with high early and continued high plastic properties.
6. Long continued gain in strength.

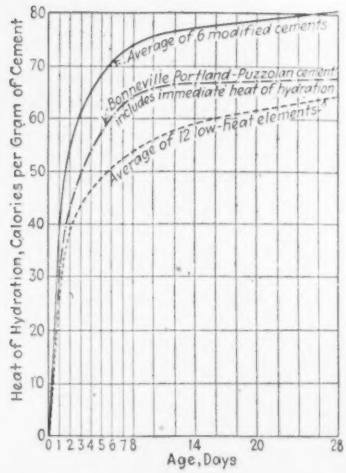


FIG. 2—HEAT OF HYDRATION, for various types of cement, averaged.

Specifications then were prepared which are unique in that they define the properties of the portland cement clinker, the puzzolan and the finished cement. It was specified that the portland cement component be of the modified type, moderately low in tricalcium aluminate. It was required that the puzzolan contain not less than 50 per cent of silica and not more than 10 per cent of lime; that it be calcined or dried and that a mortar composed of two parts of puzzolan ground to a specified degree of fineness, one part of hydrated lime and nine parts of Ottawa sand develop a compressive strength of not less than 500 lb. per sq.in. at 7 days and 800 lb. per sq.in. at 28 days. It

TABLE I — COMPARISON OF WATER GAIN AND PERMEABILITY

Type of Cement	Specific Surface (sq. cm. per gram)	Water Gain (cu. cm.)	Permeability Total Inflow Water Pressure (cu. cm.)
A	2,200	41	262
B	1,800	90	548
C	1,800	40	225

TABLE II — RESULTS OF TESTS ON HEAT OF HYDRATION

Type of Cement	Heat of Hydration in Calories Per Gram		
	3 days	7 days	28 days
A	58	75	85
B	51	61	68
C	55	67	78

TABLE III — TENSILE AND COMPRESSIVE STRENGTH AT VARIOUS AGES

Type of Cement	Comp. Strength in Lb. / Sq.In. 1½-In. Max. Agg.: Slump ½ In. to 1 In. Specimens 6x12-In. Cylinders Age				Tensile Strength of Concrete Lb. / Sq.In. 1½-In. Max. Agg.: Specimens 6x33 In. Age		
	3 days	7 days	28 days	3 months	5 days	28 days	3 months
A	605	1,620	3,300	3,825	105	265	295
B	165	525	2,635	4,090	40	225	285
C	790	1,755	3,535	3,805	115	335	350

TABLE IV — STRENGTH COMPARISONS OF THREE CEMENTS ON THREE JOBS

Dam			Bonneville	Morris		Tygart	
Type of Cement			Portland-puzzolan	Low-heat portland		Modified portland	
Mass Concrete	Water/cement ratio by weight		0.63	0.59	0.63a	0.53	0.63a
	Cement content, bbl. per cu.yd		1.00	0.95	....	1.00	....
	Maximum size of aggregate, in		6	6	6	6	6
Wet — Screened Concrete	Slump at mixing plant, in		3¼	¾		1¼	
	Compressive strength of standard cured cylinders lb. per sq. in	7 da.	1350	1150	1000	1990	1380
		28 da.	2740	2850	2480	3810	2820
		3 mo.	3810	4350	3820	4710	3580
		6 mo.	4200	5000	4450	4670	3590

TABLE V — TEMPERATURE RISE COMPARISONS ON THREE DAMS

Dam	Cement	Temp. Rise (deg. F.)
Bonneville	Portland-puzzolan	35
*Morris	Low-heat portland	40
*Tygart	Modified portland	35

*"Special cements for mass concrete" by J. L. Savage, a paper presented at the world conference on large dams.

was required that the portland-puzzolan cement be manufactured by intergrinding three parts by weight of portland cement clinker with one part of puzzolan.

The physical requirements for the finished cement with regard to time of setting, soundness, and tensile strength, were the same as those in the A.S.T.M. specifications for portland cement. A moderately high fineness of finished

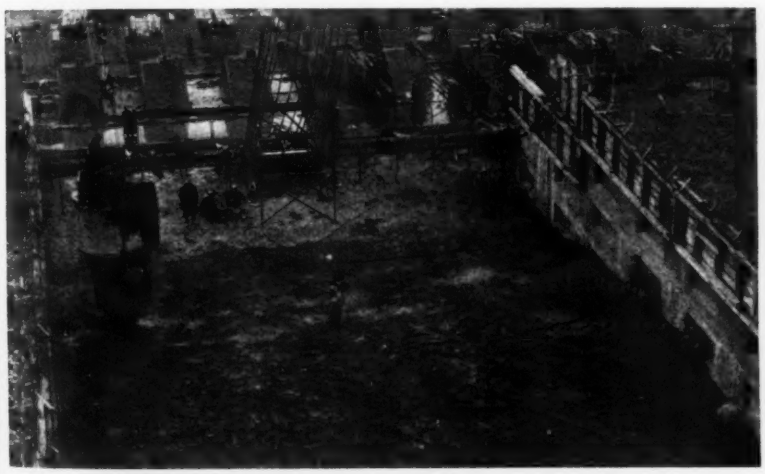


FIG. 3—PLACING CONCRETE on Bonneville Dam. Workmen are instructed to remain in the clear while the bucket comes down and discharges.

cement was specified; it was required that not more than 12 per cent be retained on a No. 325 sieve (43 microm openings) and that the specific surface as determined by the hydrometer method be not less than 1800 sq.cm. per gram. The detailed requirements of the specifications, in comparison with those for other large concrete dams, are listed in *Engineering News-Record*, Feb. 6, 1936, p. 205.

The cement, estimated at 650,000 bbl., was purchased from the Pacific Portland Cement Co. at a contract price, delivered, of $1.62 per barrel of 376 lb., corresponding to a mill price per barrel of approximately $0.75. The cement is being manufactured at Redwood City, Calif., whence it is shipped by rail to the dam site. Cement is inspected at the mill by the San Francisco office of the National Bureau of Standards. The puzzolan is a finely divided siliceous material dredged from San Francisco Bay which is calcined at about 1650 deg. F. before being blended with the clinker and gypsum.

Concreting experience

The first concrete was placed in September, 1935. Routine testing of all concrete placed includes daily sampling of all mass mixes. Soon after the award of the cement contract two series of tests were conducted utilizing the cement and aggregate to be used. As a result of these tests, mixes were adopted for both mass and reinforced sections and it was found that as low as 0.88 bbl. of cement per cubic yard of concrete produced a workable mix.

Except for the first concrete placed in 1935 with a placing temperature of about 70 deg. F. in which 0.90 bbl. of cement per cubic yard was used, the principal part of the first season's mass concrete contained 1.00 bbl. of cement per cubic yard of concrete with an average slump of 3¼ in. At the end of the first season's work and with the experience gained in placing some 275,000 cu.yd., design considerations of the second half of the dam made it appear advisable to reduce the slump to 2 in. and adopt a cement factor of 0.9 bbl. per cubic yard for all mass remaining unfinished.

Compressive strengths of concrete wet-screened from the full mass mix containing 1.0 bbl. of cement per cubic yard are shown in Table IV, at ages of 7 days to 6 months. For comparison are shown corresponding values for Morris Dam, in which a low-heat portland cement was used, and for Tygart Dam in which a modified portland cement was used. Also, for the other dams are shown the approximate equivalent concrete strengths computed for the same water/cement ratio as that used for the concrete in Bonneville Dam.

As compared with the low-heat portland cement, when the same water/cement ratio is employed, the portland-puzzolan cement shows relatively high concrete strength at early ages and about the same strength at 3 and 6 months. As compared with the modified portland cement, when the same water/cement ratio is used, the portland-puzzolan cement shows about the same concrete strengths at 7 and 28 days, but higher strengths at 3 and 6 months. With regard to the long continued gain in strength, the portland-puzzolan cement is similar to a low-heat portland cement.

As soon as cement deliveries began, tests were made to determine the heat of hydration which is shown in Fig. 2, in comparison with low-heat cement and modified portland cement. This early heat generation permits a larger portion of the total heat to be dissipated during the three days that the concrete lift is left uncovered than with a cement generating its heat more slowly. Because of difference in air temperatures, placement schedules and thermal constants of concrete, it is impracticable to compare accurately the three types of cement in different structures

through the study of temperature records in dams; however, it is of interest to note the actual average temperature rise in the three dams mentioned in Table V.

Tests for water gain made on cement delivered gave results closely comparable with that of the *C* cement of the early investigation. Since it has been shown that high impermeability goes with low water gain, it is evident that concrete produced with this cement is satisfactory so far as permeability is concerned. This is borne out by the fact that at the completion of the first half of the dam and with a water head of 80 ft., no leakage was noted in inspection galleries, except for seepage at one contraction joint not yet grouted. The puzzolan material, when interground with portland cement clinker evidently grinds to a greater fineness than the cement. This is shown by screen analysis in the laboratory. The weight of Bonneville concrete is 155 lb. per cu.ft., and is perhaps explained by this fineness of puzzolanic ingredients.

Results satisfactory

Observation of concrete operations as well as of the finished concrete work discloses nothing contrary to the satisfactory results of tests. The low strength mix, utilizing only 0.9 bbl. of cement per yard of concrete, is satisfactory from the standpoint of placement. The slump of 2 in. at forms does not result in any separation. Delivery is made in 8-yd. buckets and the concrete is readily placed with three vibrators even though delivery is made in excess of 150 cu. yd. per hour. Stripped surfaces exhibit little or no honeycombs, sand streaks or other defects.

Careful search has been made for cracks in mass concrete as well as in reinforced sections. In one of the blocks now exposed for its full cross-section, in the north cofferdam (Fig. 1), two vertical cracks were found parallel with the axis soon after the mass was placed. Each crack starts from an abrupt change in bedrock level, the principal one being the extension of a bulkhead form placed in the first lift These cracks have been measured and the maximum opening which was not over 0.03 in. is now somewhat less. In the next block, adjacent to this one, similar vertical cracks were noted in a transverse inspection gallery. They too are extremely small, and now appear to be closing.

Cooling now has progressed in the mass to form 35 per cent to 60 per cent of the total of 40 deg. F. through which it must cool, and it seems safe to conclude that this type of cement is superior for hydraulic structures in general and mass concrete structures in particular. It is also believed that the absence of cracks so commonly observed in dams, while in a measure attributable to the low temperature rise and fall of the concrete mass, is principally due to low early rigidity and at later ages plastic extension which tend to reduce greatly the thermal tensile stresses especially at the junction between lifts.

The consideration given to this property of the concrete in the original design seems justified by the results obtained. These results should be compared with preliminary studies made at a time when it was planned to use a normal portland cement. Had a cement of that type been used cracks endangering the integrity of the structure could have been prevented only by the use of contraction joints parallel with the axis.

Design and construction of the Bonneville project are under the Portland division of the Corps of Engineers, U. S. Army. Col. T. M. Robbins is division engineer; Lieut.-Col. C. F. Williams, district engineer—both at Portland, Ore. Capt. Colby Myers is resident engineer at Bonneville Dam. Civilian engineers connected with the work are C. I. Grimm, head engineer; A. E. McKennett, principal engineer; R. R. Clark, design engineer, and in the field, B. E. Torpen, senior engineer, O. C. Hartman, resident engineer and I. E. Burks, chief concrete technician.

Flood Control Being Tested On the Mississippi

Flood-control works costing a third of a billion get first test by the record Ohio River flood now flowing down the lower Mississippi

THE PRESENT great flood is expected to put under critical test the comprehensive works begun in 1928 to control floods on the lower Mississippi River from Cairo, Ill., to New Orleans, La. What are these works? How were they planned? What are they expected to accomplish? These questions are answered in concise form below by Charles S. Hill, associate editor of *Engineering News-Record.*

Project begun in 1927

In 1927 the lower Mississippi River was visited by the greatest flood of record. The river rose to unprecedented heights and remained high for an exceptionally long period. The first wave reached its crest at Cairo on Jan. 7 and the last wave crested at Carrollton, La., at New Orleans on April 25. During this period the levees were breached in 13 places; the released waters flooded 23,000 sq.mi. of land and drove hundreds of thousands of people from their homes. Beyond its cataclysmic aspects, which were overemphasized, this flood had epochal influence on Mississippi flood control policy: Its floods were accepted as a national responsibility and a comprehensive plan of flood control was originated.

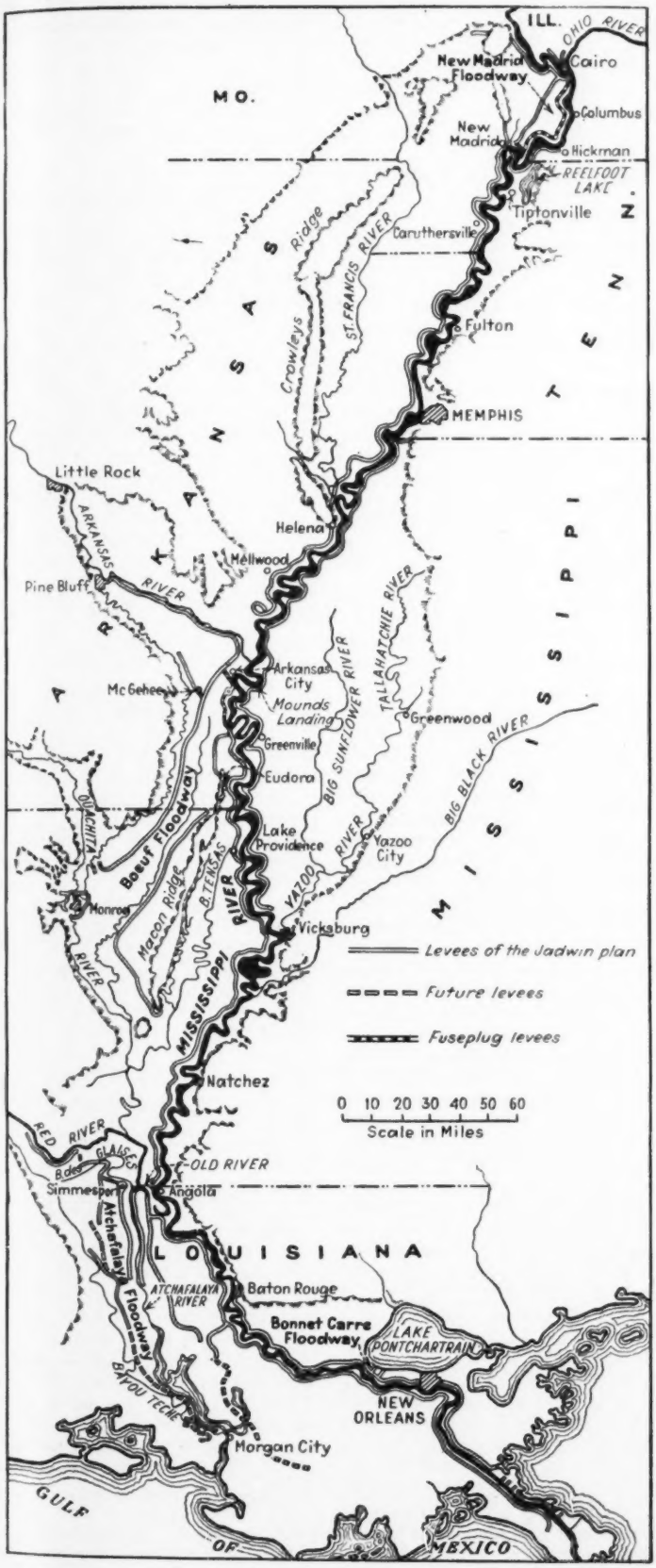
Demands on Congress for action to control floods immediately came from the states that had suffered damage in 1927. They fell upon a sympathetic national legislature and in general a sympathetic public. Committees were appointed by Congress to draft a national flood-control act for the lower Mississippi River. The committee of the House of Representatives called many hearings and discussion waxed furious over plans, division of costs and scores of minor details. Out of the confusion one sure thing was apparent: National assistance in flood control was certain to be extended to an extent unprecedented in the United States and perhaps in the world.

The first requirement for legislation was a plan for flood control that could be written into law. A number of plans were offered, earnestly advocated and vigorously opposed. The two leading ones, remarkably similar, were proposed by the Corps of Engineers U.S.A., with Maj.-Gen. Edgar Jadwin as Chief of Engineers, and by the Mississippi River Commission with Col. Charles L. Potter as president. Both plans threw overboard the long-held doctrine of flood control by levees alone, and advocated a plan of relief floodways to draw off and divert the flood flow that could not be carried between levees reasonably heightened and strengthened beyond those disclosed inadequate by the 1927 flood.

Rival plans

With the submission of the Jadwin and the Commission reports on plan, Congress found itself with rival proposals on its hands and the task of making a comparison and of selecting the better plan. Chiefly the two plans differed in the total volume of flood flow and in the relative proportions of this flow to be carried between the main river levees and by the flood ways.

The Commission plan provided for higher levees and a spillway below New Orleans but did not provide for



LEVEE SYSTEM of the Jadwin plan for control of floods on the Mississippi River.

a floodway below Cairo, leaving the relief of this critical section of the river for further study and later planning. The Jadwin plan did not include a spillway below New Orleans but did provide definitely for a floodway below Cairo. It provided like the Commission plan for raising the levees, but not to so great a height, and like the Commission plan called for diversion floodways down the valley of Bayou Boeuf, just below the mouth of the Arkansas River, and down the Atchafalaya River, just below the Red River, and for a controlled spillway at Bonnet Carre just above New Orleans.

The Commission plan provided for controlled spillways at the Boeuf and Atchafalaya floodways as well as at Bonnet Carre; instead the Jadwin plan provided for "fuseplug" levees at the floodway entrances at the Boeuf, the Atchafalaya and at the Birds Point diversion at Cairo. The fuseplug plan and theory are in brief as follows: Above and below the floodway entrances the river levees are raised (about 3 ft. in the Jadwin plan), but in front of the entrances they are left at the old lower height or cut down somewhat. When the flood rises so that diversion is necessary the water overtops the section of low levee, washes it away and a part flows down the floodway away from the river.

With the two plans before Congress for choice, a long discussion began. Both plans had active advocates and critics in Congress and outside among engineers and the various river interests. The discussion at the Congressional flood control committee hearings was furious and often acrimonious. Opinions were radically opposed on the sufficiency of the Birds Point floodway, on the hydraulic efficiency of all the floodways, on fuseplug levees, on compensation for floodway land and on a score of other engineering and financing questions. A bewildered Congress floundered in the morass of confusing ideas and beliefs.

Then there was the cost problem. The Jadwin plan was estimated to cost $296,400,000 and the Commission plan to cost $775,000,000. The difference was accounted for by the far greater volume of levee construction called for in the Commission plan, by the greater cost of controlled floodways, and by payments allowed as damages to floodway lands. Congress shied at the greater cost figure, the assumption of flood control by federal government was a new and somewhat feared venture, the executive pressure was to keep cost down, there was strong opinion that increased protection was enough for land owners and that use of their land for occasional flood service ought not to be paid for heavily. All of these considerations had great influence, probably much greater than any engineering considerations.

Congress finally settled on a flood-control act which embodied almost completely the Jadwin plan, and this act was made law on May 15, 1928. The act authorized the appropriation of $325,000,000 to carry out the adopted plan, but $15,000,000 of this was for previously authorized construction and for emergency flood work. Besides providing for the physical work required by the plan the law set up a board to consider the differences between the adopted and the Commission plan and report to the President. A change in administration was also provided: The Mississippi River Commission, formerly an independent administrative body reporting directly to the Secretary of War, was placed under the direction of the Chief of Engineers and made an advisory body with certain powers to take evidence and recommend action. The remaining step before final decision on plan was the report to the President of the board of review authorized by the flood control act. This board consisted of Major General Edgar Jadwin, Brigadier General Thomas H. Jackson, Corps of Engineers U.S.A. and Carleton W. Sturtevant, consulting engineer in private practice. The report made in the summer of 1928 recommended the official (Jadwin) plan in virtually every detail. It placed the total estimated cost of the work at $318,000,000.

Official (Jadwin) plan

As developed the plan contemplated (1) the raising of the levee line by about 3 ft. above the previously adopted or 1914 grade line, along the whole length of about 1,000 miles of river from Cairo to near New Orleans, to carry a flood ranging from 2,250,000 sec.-ft. at Cairo to 3,000,000 sec.-ft. in the lower river; (2) the construction of an emergency spillway at Bonnet Carré, just above New Orleans, capable of discharging 250,000 sec.-ft. of flood flow into the upper end of Lake Pontchartrain (an arm of the Gulf of Mexico); (3) the construction of three wide floodways outside the normal leveed width of the river's high-water channel, one just below Cairo (Birds Point floodway) to lower the flood crests at Cairo, a second from the mouth of the Arkansas southerly through the westerly half of the Tensas Basin (Boeuf floodway) to carry the excess floodflow which the leveed river apparently could not accommodate, and a third along the course of the Atchafalaya River, to carry 1,500,000 sec.-ft. of water from the Red River and from the Boeuf floodway discharge directly to the Gulf, leaving only the remaining 1,500,000 sec.-ft. to go down past Baton Rouge and New Orleans.

In addition to these specific construction operations, stabilization of the river channel was to be carried out. Methods of stabilization beyond the customary bank revetment were not set out but were left to be developed by subsequent studies and experience.

At present the main river levees, except at floodway entrances, have been raised to project grade; the Birds Point floodway is completed; no construction has been carried on in the Boeuf floodway except to complete a ring levee around Monroe, La.; in the Atchafalaya some work has been done on the guide levees and more on the river levees, and a large amount in rectifying the river channel by cutoffs and dredging; the Bonnet Carré spillway has been completed.

Outstanding progress has been made in channel stabilization of the main river. There has been extensive construction of timber and sand spur dikes and of bank revetment and a remarkable straightening operation by cutting through oxbows and bends. Going south from the Greenville Bends twelve loops of the river have been eliminated, shortening the channel over 100 miles.

Expected results

These are the flood control works that the present superflood from the Ohio River promises to test. They have never been tested by flood. What the results will be time has yet to tell. Meanwhile it is interesting to review what the test was that was planned for and what results were expected.

By a computation never clearly indicated the Jadwin plan assumed a superflood, to resist which the control works were to be designed. Specifically the volumes of flow to be provided for were: At Cairo 2,250,000 to 2,400,000 sec.-ft., of which the Birds Point floodway was expected to take 400,000 sec.-ft.; at Arkansas City, 2,850,000 sec.-ft., of which the Boeuf floodway was expected to take from 900,000 to 1,250,000 sec.-ft.; at Red River landing 3,000,000 sec.-ft., of which the main river was expected to take half and the Atchafalaya the other half; and at Carrollton, La. (New Orleans), 1,250,000 sec.-ft. not including 250,000 sec.-ft. to be diverted by the Bonnet Carré spillway. Considering the river as a whole this assummed probable flood was about 25 per cent greater than the record flood of 1927.

On the supposition that the discharges would be as indicated the levees were designed with a free height above water (freeboard) of 1 ft. This margin of safety was widely criticized as being inadequate but it was maintained generally in the construction. It was also contended by critics that the assumed discharges and levee heights did not provide for possible local piling up of flood peaks that, when leveled out down river, were of little general flood importance..

The floodway provisions of the control plan aroused wide and persistent contention. Assumed capacity of diversion, adequacy of fuseplug action, and justice to floodway land owners were questioned. Mention of the Boeuf floodway will call to mind the controversy that raged around this part of the project.

Floodway doubts

The people who live in the strip 10 or 12 miles wide and 150 miles long, which was to be inclosed within the floodway guide levees, feared that the value of their property would be destroyed without recourse, as the law made no provision for compensation to be paid them. They applied for an injunction to restrain the building of the floodway levees, but finally lost in the U. S. Supreme Court on legal grounds, the decision making no finding as to whether these people would suffer damage from the construction of the floodway. Because of this controversy no work has been done on the floodway.

The experience at the Boeuf reacted on the plans for active prosecution of the Atchafalaya floodway work. In effect work was abandoned so far as the original plan definitely provided. Later it was resumed, but as a work of channel rectification and enlargement coincident with stress put on similar operations on the main river.

But while land disputes were largely responsible for removing the Jadwin design of floodways from the diversion plan, this design had many questioners on technical details. The fuseplug-levee gateways to the diversion channels, it was contended, were not certain to fuse, might not fuse fast enough and could not be closed when the flood had fallen below the level at which diversion was required. So also the broad floodway channels, remaining forested and in other condition of nature, were held to be highly uncertain conduits for the getaway of water in such vast volumes. At the Birds Point floodway engineers are now getting the first test data on fuseplug levee, flood getaway and backwater action.

In case of individual floodways, criticism of sufficiency of the guide levees was extensive. Length, location and elevation of these embankments were claimed inadequate in instances to protect the backcountry lands and settlements. At the Boeuf and at the Atchafalaya the change of plans make these questions of little present interest, but below Cairo they are being answered as the flood water laps at the top of the setback levee and New Madrid, Wyatt, East Prairie and Charleston watch it anxiously.

The crest of the present flood has passed Cairo. What may be expected farther down the river? The answer depends on the continuation down the river of the high stage at

Cairo. If this is proportionate to past flood experience a real test of the sufficiency of the 1928 levee grades is indicated.

The second critical point is the valleys of Bayou Boeuf and the Tensas River, the Boeuf valley being the location of the originally planned floodway. No construction, as before stated, has been done here: the old 1914 levee at the floodway entrance remains at its original height some feet lower than the rebuilt levees (1928 grade) above and below. The obvious danger point is the old levee, which if overtopped and breached will roll floodwater down the Boeuf and Tensas as unrestrictedly as in the old days.

The natural procedure will be to delay overflow of the old levee as long as possible by raising the embankment by emergency bulkheads as was done at Cairo and in other places where thousands of men are now working. Action in this respect will be controlled partly by critical conditions occurring elsewhere.

Farther south and 200 miles above New Orleans is the next relief valve at the Old River entrance to the Atchafalaya. As stated, the Atchafalaya has been straightened and the capacity of its flood channel otherwise increased. If these improvements prove incapable of carrying the flood overflow from the main river, the old levees (fuse-plug) will breach and the valley will be inundated as in the past. Fortunately for flood safety at the Boeuf and the Atchafalaya, the upper Mississippi and the great western tributaries—the Arkansas and the Red—are at moderate stages.

The final interesting point of the flood attack will be the Bonnet Carré spillway just above New Orleans. As the only completed relief valve of the original plan, except the Birds Point floodway, and as the only controlled spillway and moreover one of immense size, Bonnet Carré is of exceptional interest should it be called upon to take its planned diversion of 250,000 sec.-ft. from the river. With a concrete sill and apron over 7,000 ft. long founded on piles in silt and a take-away floodway in silt the Bonnet Carré in action presents questions of foundation stability, scour and sedimentation of vital engineering interest.

About the new plans

Thus far consideration, except incidentally, has been confined to the Jadwin plan structures. Incidentally this plan called for channel stabilization, but as a secondary matter. In 1932 stabilization took definite form as a major flood-control operation. In a measure the new activity was directed toward avoiding floodway land controversy such as had stopped work on the Boeuf floodway, but there was also a more fundamental inspiration. It was believed possible to increase the capacity of the river as a hydraulic conduit by straightening the channel and improving the bottom slope, and this work has been pressed since 1932. Numerous spur dikes have been built, points of bank scour have been revetted, a dozen river loops have been eliminated by cutoff channels, and kinks straightened out in the flood channel levees.

Regardless of floodways, a river of hydraulic characteristics quite different than in 1927 is presented for trial by the present flood. The action will be of intense interest to river engineers.

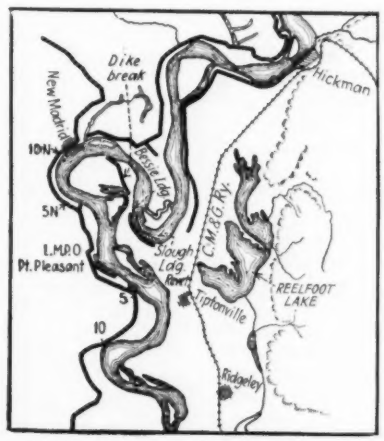
Fighting High Waters

First-hand observations of the battle to control the superflood now raging in the Mississippi Valley

By H. W. RICHARDSON
Associate Editor, Engineering News-Record

On Board U. S. Engr. Dept. Boat "Petrel" Enroute Porter Lake, Ark., to Memphis, Feb. 2, 1937

THIS IS WAR, a relentless, unceasing and determined battle of man against the equally persistent and relentless Mississippi River raging down the valley from Cairo to the Gulf in its greatest flood in history. The river is trying to claim its own—the broad alluvial valley that man has moved in to occupy — while man is fighting hard to keep the superflood confined by hundreds of miles of slender earth dams.

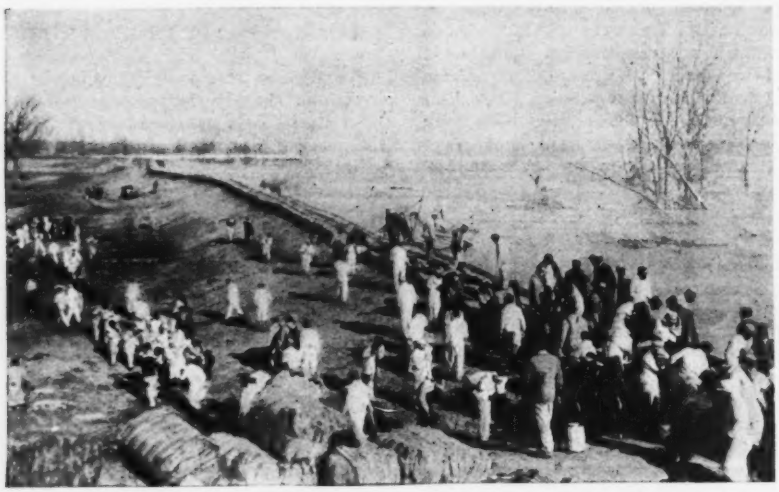


THE RIVER is cutting across the bends near Reelfoot Lake, having topped the secondary levees at Bessie Landing. A hard fight is being made to hold the main levees on the Missouri side.

The river won by overwhelming odds in the Ohio valley, catching his enemy wholly unawares and engulfing his proud cities to unprecedented depths, leaving misery and destruction in its wake. However, the river is losing the fight so far in the second salient of this war, the Mississippi Valley. Radio reports this morning state that the crest of the flood is passing Cairo, and the city still stands unharmed, while below Cairo the waters are being successfully confined to the channel outlined by the levees of the 1928 flood-control project. Reports from all along the front grow more assuring hourly as low spots in the levees are raised to safe limits. The army engineers, veterans in many fights with the mighty Mississippi, are now confident that for the first time a major flood (and this is the greatest flood of the Mississippi's history) will be carried safely to the sea without a serious break in a main levee. In the great flood in 1927 the river breached the levees in scores of places and poured over millions of acres of bottomlands.

To understand the battle that is now being waged in the Mississippi Valley it is necessary to know something of the battlefield. The Jadwin project, planned and built after the 1927 disaster, provided for enlarging and raising the main-stem levees to carry off a flood of 59.5 ft. stage at Cairo and 54.5 ft. at Memphis (the latter being the U. S. Eng'r. Department gage, 1.6 ft. higher than the Weather Bureau gage commonly quoted). In addition, the plan as finally adopted included floodways outside the levee lines: One from Birds Point to New Madrid, Mo., designed to decapitate the crest at Cairo; one through the Boeuf-Tensas basins for relief of the Arkansas River reach; one through the Atchafalaya basin for handling the Red River; and the controlled Bonnet Carré spillway into Lake Pontchartrain for the protection of New Orleans. At the Birds Point floodway a new inland levee restrains the flood, guiding it back into the river 35 mi. below the "fuse-plug" opposite Cairo.

When the Ohio went on its rampage, the engineers responsible for the Mississippi control became scared, and with all good reason. All indications pointed to a flood stage of 62 at Cairo with the Birds Point floodway in full operation, and to a river height of 55 at Memphis, 8.5 ft. above the previous record of 46.5 in 1913. It was apparent that the Jadwin-project height of levees would be overtopped by a considerable margin. Consequently, war was declared on the



LEVEES ARE BEING RAISED in the Porter Lake region from 3½ to 4 ft. in anticipation of a rise in the river level above the project grade.

flood, and the entire front of the Memphis engineer district, from Cairo to the Arkansas River, was attacked. Plans were immediately made to raise the levees where necessary to cope with the expected flood heights, and this is the battle that is being waged today. What is being done downstream from the Memphis district limits is another story.

For a year now the Memphis district engineers have been making preparations for this superflood. They didn't know just when it was coming, but they were certain it would come some day. Consequently, the campaign of the present fight was already planned. The first step was to make accurate profiles of the existing levee grades, and plot on them the theoretical project grade. When the levees were enlarged and raised, 25 per cent excess material was placed to allow for a settlement. This subsidence and compaction has been very uneven. In places the profiles show that the embankments remain as much as 4 ft. above project grade, while at other points the subsidence has dropped it below the desired height.

When it was determined to make the levees safe for the current flood, calculations of flow indicated that the project grade must be raised to new heights, varying from 2 ft. on the Birds Point floodway guide levee down to 3 in. on the main embankments farther downstream. These grades were plotted on the levee profiles, and at once the engineers knew what sections required raising. The old-time civilian engineers attached to the district—the men who stay year in and year out while the army engineers are shifted around—were also called on for their knowledge of weak spots in the line.

First hand observations

Determined to find out just how this war is being waged, I called at the Memphis engineer district headquarters in West Memphis, Ark., just off the west approach to the Harahan bridge, the only highway outlet from Memphis to the west. Long before the office was reached signs of most unusual activity were apparent. My cab was stopped a dozen times by police and highway guards. No sightseers were wanted; through travelers were warned of road conditions ahead, where every route was flooded by backwaters from the St. Francis. Trucks swarmed around us, headed for the engineer depot loaded with all kinds of supplies—for this army must have supplies—and sacks. I never dreamed there were so many sacks in the world. Burlap bags, such as are used for sugar and potatoes, are preferred, and some 7,000,000 have already been purchased by the Memphis district. Activity increased as we neared the district headquarters. Highway police patrolled the road to keep traffic moving, but in spite of their efforts jams resulted from the multitude of supply trucks trying to work through herds of mules and cows being driven out of the west side lowlands to higher ground around Memphis.

Though the district headquarters, on a point of land, are well protected by a concrete seawall, a 4-ft. mudbox is being built on top of the wall for further protection. Boats swarm about the docks, every conceivable kind of boat known to inland waters: outboards, dredge tenders, launches, small and large towboats, survey boats, quarterboats, coast-guard cutters and even navy officers' barges. These boats have been called in from all over, from as far up the river as Kansas City, from New Orleans, Jacksonville, and even from New England. Gloucester fishermen on the Mississippi are indeed a strange sight.

Entering the main office, I found the headquarters telephone exchange, temporarily moved from the first floor to the hallway of the second floor; the engineers are taking no chance of having their vital communication facilities flooded out. The sharp staccato of the Morse code came from the temporary radio room, where the signal corps is keeping in touch with short-wave portable sets at field stations throughout the 400-mi. length of river in the district.

In the halls and offices I met many an old friend called in from engineer corps projects from far and wide. The entire Conchas dam organization, headed by Capt. Hans Kramer, is here; Kansas City, Jacksonville, Huntington, Pittsburgh, Galveston and other districts furnished officers and men for this fight. Col. Eugene Reybold, Memphis district engineer, is the general in command. Ft. Belvoir Engineer School must be deserted, for I met dozens of officers from there.

A levee-raising job

So I wanted to see a levee raising job, did I? One of the biggest jobs was at Porter Lake, Ark., 30 mi. downstream, and a boat was leaving at once; if I cared to join the party enroute there I was welcome. Armed with an official pass, but without baggage or even a shaving kit, I shoved off at noon in the motor boat *Pointer* with eight foremen and superintendents. The *Pointer* is a small gas boat used for survey work in the Kansas City district. None of its crew had been on the Mississippi before, and with some misgivings we cast off into the turbulent, swift-moving river in a gale that was whipping the long reaches into high white-caps. I had pocketed a few index maps of the district, and from those small prints we navigated safely, though roughly, down the river to Porter Lake, often taking through the tree tops to cut across a long bend.

Three hours out of Memphis we tied up alongside U. S. E. D. Quarterboat 2208, and all breathed a prayer of thanks for getting down safely. Here was a scene of activity never witnessed on any construction project. Over 600 negro convicts were laboring to raise a 4,100–ft. stretch of levee by sandbagging 3½ to 4½ ft. above original grade. This particular section was an old levee looping around in front of a brand-new stretch of hydraulic-fill embankment in the main levee system. The hydraulic section is high enough to withstand the flood, but it is so new, so untried, that the engineers preferred it be given another year of consolidation before subjected to a super-test.

The gang was just finishing up the 4,100 ft. of raising when we arrived. They had been on the job less than three days. Wooden bulkheads of longitudinal 1-in. boards held by 2x4 and 2x6 posts had been erected on the river side of the top of embankment, and where the levee became a spur out into the river bulkheads had been placed on both sides.

Building up of the sandbag topping was by a never-ending stream of carriers who transported half-filled sacks, weighing about 75 lb. each, to waiting placers and tampers at the top of the levee. Scores of convicts worked in pairs filling the bags far down the rear slope of the levee. The front wall of sacks, against the bulkhead, is built vertically; the back side of the topping is stepped down. Usually the base row of sacks is made twice as wide as the height of topping wall. Sacks are placed alternately as headers and stretchers, with the loose ends carefully tucked under adjacent bags. It is important, I learned, that the bags be not too full, to permit squashing and tamping into flat, even layers. As the topping progresses an occasional bag is dumped and its contents tamped into the voids between bags. This procedure gives a remarkably tight wall. Bags are tamped into place by workmen's feet and by heavy wooden pounders, the same as used for consolidating trench backfill.

SAND BOILS behind the levee are ringed with a sandbag wall to reduce the head and check erosion.

Different kinds of river work

It was a bitter cold day, with the wind sweeping full force down the levee. Trusties armed with shotguns huddled around fires at extreme ends of the job. The convicts, fairly well clad, hustled to keep warm. Never have I seen men work any harder, and checking up later I found that this crew had placed more sacks per man per day than any other gang on the river. They go to work at daylight, work until dark, though checking in and out and stopping for the noon meal cuts down actual working time to 7½ or 8 hr. The convicts are housed in covered cement barges anchored near the job.

The organization at Porter Lake is typical of that of levee raising in the entire Memphis district. Capt. Hans Kramer is area engineer in charge of this part of the river. The quarterboat is field headquarters for a subarea embracing 26 miles of main levee. Blair A. Ross, George Little and H. J. Baker, all civilian engineers and veteran levee men, are in charge of the sub-area. Lieut. H. G. Davis, on temporary leave from Ft. Belvoir, is the army engineering representative. A hand-cranked portable radio set, manned by a member of the army signal corps, is the sole contact with headquarters at Memphis outside of boat dispatches. Access by land is entirely cut off by backwater.

In this 26-mile sub-area, besides the 4,100-ft. section of raising just described, is a 14,000-ft. section that requires only a couple of layers of sandbags, as the grade is within 3 in. to 8 in. of that desired. Also, at the north end of the area at Blue Lake an old levee is being cut to allow waters to fill against a new section built farther back. This is being done in many places, I find, to permit the new embankments to get the feel of water ahead of the flood.

Boils are a constant menace, and the Porter Lake section has its share of this trouble. Seepage usually breaks through at the rear toe of the levee. If the boil runs clear water it is not regarded as particularly dangerous, for experience has shown this type usually plugs itself up. However, if the boil carries soil and sand, steps are taken at once to stop it, for this is a sign the bank is being undermined. The treatment is to build a ring of sandbags around the seep high enough to build up the hydraulic backpressure sufficiently to counteract the outside pressure. These boils are carefully watched for further trouble.

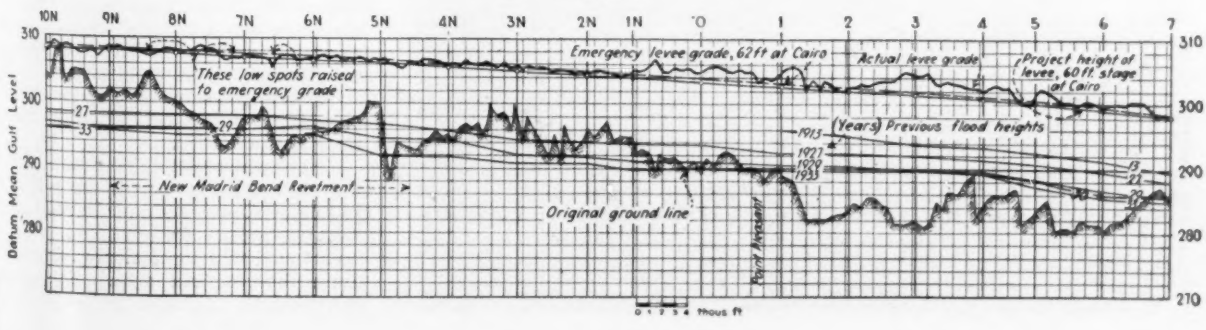
The entire levee system is being patrolled 24 hr. a day by experienced levee workers. Each man has from 1 to 2 mi. of levee to watch, and army field telephones are being installed for communication with field headquarters.

Other types of topping

In talking with the supervisory force I learned something of the different methods employed in emergency raising of levees. The work in the Porter Lake District is typical of sandbagging. Bulkheads on the front side are used wherever wave wash is threatened and where time permits their installation. Sandbags alone are used for low-height topping and in fast emergency work where raising proceeds just ahead of the rise of the river.

Another very satisfactory method of raising is the mudbox, the construction on which Cairo relied for fortifying its Ohio-side levee. This comprises two rows of wooden bulkheads tied together at the top by wood struts cross-pieces and filled with bulk material thoroughly tamped into place. It is more premanent than sandbagging, for the burlap bags soon rot and the wall disintegrates. Where possible, the mud-boxes are filled by wheelbarrows run

CRITICAL SECTION OF LEVEE south of New Madrid. (See map on p. 225.) The low points are being raised by emergency measures. The numbers at the top are the local levee mile posts.



along plank runways on top of the crosspieces.

Vertical sheeting is sometimes used in front of sandbag walls, especially in weak spots threatening to break through. However, in the main sand bags are preferred, for every worker, every native and every levee man is familiar with their use. They are simple to use, require no skilled help or engineering, and form a wall easy to repair.

When time permits, that is, when the topping is well ahead of the river, the top grade is established by a survey crew carrying levels from benchmarks established at levee mile posts. When time is short or when no survey instruments are at hand the crews work to grade heights above river levels. It is known how much freeboard is wanted, and with daily river stages known it is a simple matter to carry the topping to a definite height above daily river levels.

At Porter Lake all supplies have to be brought in by boats and barges, and it is up to the field superintendents to notify headquarters of their needs far enough in advance to have supplies shipped from Memphis. In other locations supplies are hauled in by truck or rail, where such transportation facilities are available.

Right now I am returning to Memphis on board the small diesel pushboat "Petrel." We are following up the riverside borrowpits along the levee, where possible, to escape the current sweeping down the main river. Now and then we get hung up in the tree tops, but the operator gets us through somehow with remarkable skill. We have a sick convict with us bound for the Memphis hospital. He has an ulcerated tooth, with a jaw like a pumpkin, and displays little interest in his surroundings.

Such is the fight being waged on 560 miles of levee along the river. It is a war to the finish, and all indications are now that the main levees will hold.

Seldom do the engineers and supervisors have a moment to relax. Last night was one of those rare moments, and a poker game was soon started among a group huddled around the stove in the quarterboat, shivering every time the wind whipped the boat into the bank.—Oh well, I suppose it is worth something to learn that aces don't stand up on the Mississippi as well as they do on some tunnel jobs I have covered.

For further reports see the news pages of this issue.

Stage Construction Employed For St. Louis Streets

Current street design is looked upon as a growing process to keep abreast of new advances in materials research and progress in traffic practice

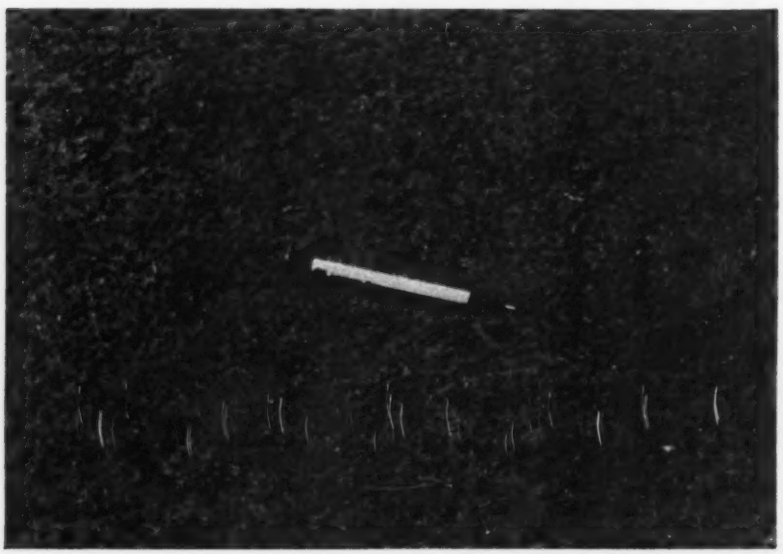
By L. A. Pettus

Division Engineer, Sewers and Paving Section, Board of Public Service, St. Louis, Mo.

MUCH MORE than ever before is now demanded of city streets. The advent of the motor vehicle and its accelerating rate of improvement and the rapid developments in pavements constantly press new problems on the street engineer. Changes in practice must be frequent to meet changing traffic conditions. Street surface design then is not something that can be fixed except partly for certain fundamental elements as foundations, drainage and cross-section and intersection profiles. Beyond these elements every street modernization design is a separate study in which location, traffic conditions, materials development and construction technique are considered.

The following account of street engineering practice in St. Louis, Mo., is then to be read with the clear understanding that it is merely a statement of current practices. For clarity general design elements and pavement types are discussed separately. As to the future, any one guess is as good as another. The present attitude is to construct bases today, using them for the riding surface and expecting to place a permanent wearing surface when the base needs repairs. This stage construction is an accepted practice in roadbuilding that street engineering may very properly consider.

FIG. 1—NONSKID SHEET-ASPHALT produced by rolling trap rock chips into the sheet mixture to produce a surface texture like asphaltic concrete.



General design elements

The usual widths of streets in St. Louis are 60, 80 and 100 ft. and the paving areas between curbs are limited to 36, 56 and 76 ft., respectively, thus leaving a width for sidewalk of 12 ft. between the curb and the property line on each side. These roadway widths provide for one parking lane on each side and two, four or six traffic lanes.

The width of pavement in the sidewalk area varies according to the locality and the expected amount of pedestrian traffic, but is a minimum of 5 ft. Where the sidewalk pavement does not cover the entire area, a planting space about 2 ft. wide is left between the walk and the curb. This space is also utilized for street-light standards, various traffic signs and utility service poles when these are permitted.

The planting area slopes upward from the curb ½ in. per foot, the sidewalk paving ¼ in. per foot and the remaining sidewalk area ½ in. per foot. It has been found that these slopes adequately drain the sidewalk area. The street pavement is parabolic in section, the

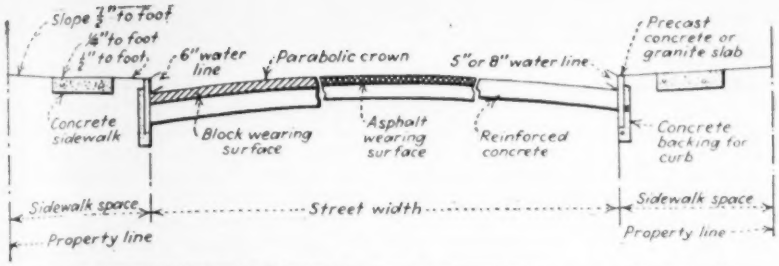


FIG. 2—ACROSS-STREET PROFILE of three general types are employed in the design of pavement in St. Louis, Mo.

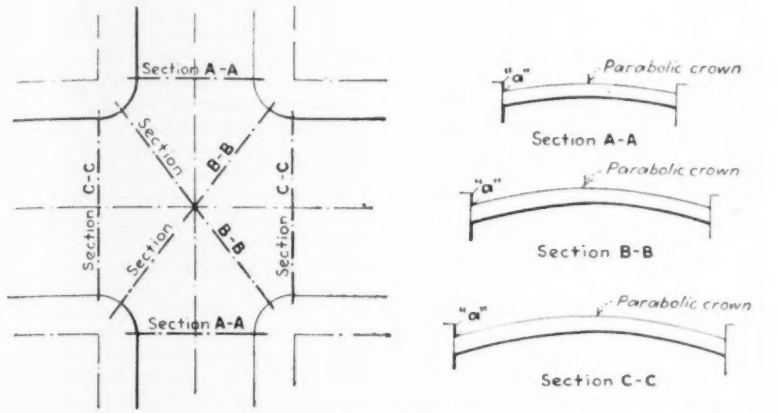
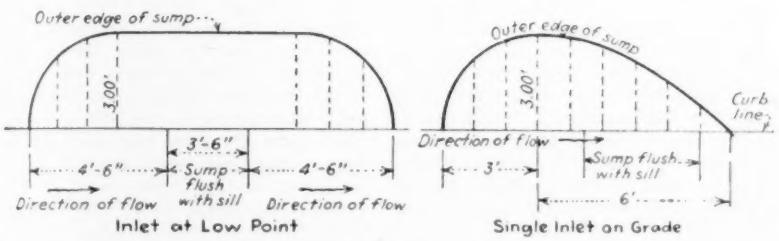


FIG. 3—INTERSECTION PROFILE designed to permit turns or straightaway travel without conscious change of grade.



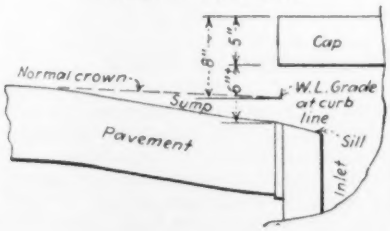


FIG. 4—OPEN CURB INLETS of two general types are installed at low point and on grades.

height of crown varying according to width, from 6 to 9 in. The usual height is about 6 in.; but this is deviated from at times according to the conditions of the particular project; where the construction is considered as a future base the curb height is increased to 8 in. Fig. 2 illustrates the sidewalk area, curb and parabolic street section as described above.

Intersections—Intersections are so constructed that the center lines of the two intersecting streets are a continuation of the approaching profiles, there being no dip or rise as the intersections are approached. The diagonal lines (Section B-B, Fig. 3) are true parabolas as is the section at the beginning of the curb rounding (Section A-A and C-C, Fig. 3). All sections between these points are warped so as to give a continuous smooth surface nearly parabolic in section.

The design described produces an intersection that permits the vehicle either to go straight through or to turn the corner without any change of grade being felt by the rider. It also keeps the run-off water confined to narrow limits so that pedestrians may cross the gutter with ease. The surveyors find these intersections somewhat complicated to lay out, but contractors have no difficulties in constructing them. It has been found necessary, however, to construct all intersections by hand methods, as finishing machines are not adaptable to the warped surfaces.

Vertical Curves—No standard methods have been adopted in determining the length of vertical curve, the decision is left entirely to the designer. On major thoroughfares all grades are reasonably light and therefore vertical curves are naturally long. On some minor streets where rapid changes of grade are necessary, it is not unusual to allow about 100 ft. for the complete change of grade. In the design of viaducts where a vertical curve is used for the change of grade, the length frequently allowed is 20 ft. for each per cent change of grade algebraically.

Grades—The terrain of the city and vicinity is rather hilly and some of the streets have grades of about 15 per cent, but on any street that is likely to have reasonably heavy traffic, an attempt is made to keep the grades below 10 per cent, and on major thoroughfares the grade is seldom as great as 6 per cent. Where two streets intersect, one having a steep grade, it is not infrequent to hang the cross street by changing the crown, the amount of change depending upon the grade of the streets and the designer's judgment.

Inlets—The open curb inlet with a sump (Fig. 4) is considered the best method for removing storm water from the streets, but in places where this design is not feasible the grate inlet is used. As the sewer system handles combined storm and foul-water all inlets are trapped (Fig. 5).

Pavement types

St. Louis uses both reinforced and plain concrete; brick and granite block pavements, and asphalt pavements, including penetration macadam, asphaltic concrete and, primarily, sheet asphalt.

Concrete—The concrete pavements are of three types: (1) The two-lane artery in unpopulated areas where curb and gutter are not constructed until future development requires a widening of the pavement. This pavement is usually reinforced concrete with thickened edges, the outer 2 ft. being 9 in. thick, the next 2 ft. varying from 9 to 7 in. and the remainder being 7 in. thick. This pavement has a longitudinal joint down the center and transverse joints about 40 ft. apart; (2) A uniform-section reinforced-concrete slab 7 in. thick with transverse joints at 40-ft. intervals, and (3) A uniform-section plain concrete constructed as in Fig. 2, except that the thickness is 7 or 9 in., depending on light or heavy traffic conditions.

Block—Block pavements are constructed on plain concrete bases 7 in. thick with transverse joints at 40-ft. intervals. The granite blocks vary from 5 to 8 in. in length, from 3 to 5 in. in width and from 4¾ to 5¼ in. in thickness. The blocks are substantially rectangular on top and sides and uniform in thickness; the sides and ends are so cut that the joints between blocks when laid are not more than ¾ in. and the wearing surface of the block has no depression

greater than ¼ in. when a straight edge is laid in any direction and parallel to the general surface of the block. While block widths are permitted to vary between 3 in. and 5 in., these widths are grouped into four sizes, so that widths in each group will not change more than ½ in., and then each size is laid in a separate portion of the street.

The bedding course is a clean crushed limestone having a specific gravity of not less than 2.56. The screenings all pass a ⅜ in. mesh and not more than 10 per cent pass an 80 mesh and they are uniformly graded between these limits. The mastic filler consists of equal parts of asphalt and sand, heated separately and mixed immediately before applying.

The blocks are laid perpendicular to the curb line and no course deviates more than 1 in. from a straight line. The blocks are laid as closely as possible, each block touching the adjoining one on sides and ends; all joints are broken at least 3 in. Immediately after the blocks are laid, clean gravel of a grade commercially known as roofing gravel is swept over the surface and into the joints to produce a stable surface for rolling. The surface is then rolled, care being taken to cross roll and diagonally roll so as not to disturb the vertical position of the blocks. After rolling, any gravel appearing in the upper portions of the joints is raked out.

After the surface has been thus prepared, the mastic filler is brought to the pavement in wheeled vehicles, with automatic means of agitation, which will permit it to be placed on the pavement surface at such rate as may be ordered. It is poured on the surface in small quantities and immediately forced into the joints by rubber-edged squeegees. The surface is squeeged as clean as possible, the intent being to leave no material on the upper surface of the blocks.

As soon as the filling of the joints has been satisfactorily completed the pavement is covered with hot torpedo gravel, graded between ⅜ in. and a No. 10 screen.

In constructing brick pavement, vertical-fibre lug brick laid on a concrete foundation is specified. The brick are laid according to the latest accepted practice, that is, on a sand-mastic cushion with a mastic filler flushed over the entire surface, after which the excess filler is removed.

Bituminous Macadam — For several years St. Louis did not construct penetration macadam streets, but during the Works Progress Administration many minor residential streets have been so surfaced, since the type is well adapted to WPA labor and meets all of the necessary requirements on minor streets. These pavements have a sledge-stone Telford base 8 in. thick, and a 2 to 2½ in. penetration macadam surface. They are penetrated with cutback asphalt, asphalt emulsion or straight 85-100 penetration asphalt. The use of custom types of asphalts depends on weather conditions and the time of year.

Asphaltic Concrete—In the past, a considerable amount of asphaltic concrete was used in new construction; recently it is much used in resurfacing various types of pavements, and at the present time the city has many streets resurfaced with asphaltic concrete. This resurfacing is saving the city many dollars yearly in maintenance and at the same time is providing desirable riding qualities.

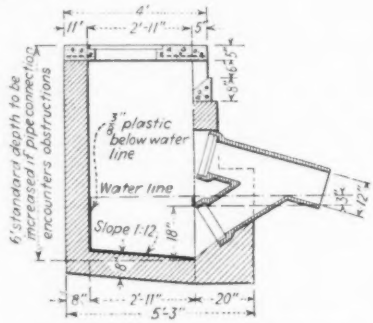


FIG. 5—TRAPPED INLETS of this type are used where foul and storm-water flow are combined.

With few exceptions major thoroughfares have been paved with hot sheet asphalt laid on a concrete foundation. The surface consists of a 1½ in. binder course and a 1½ in. top course. The binder course is composed of asphaltic cement, limestone chat, sand and limestone coarse aggregates. The sheet top or wearing surface is composed of an asphaltic cement, fine limestone chat, sand and mineral dust. The bitumen content varies from 5 to 6 per cent in the binder and 9 to 12 per cent in the surface mixture. The gradation of the aggregate varies from a No. 10 screen down to 10 to 18 per cent passing a 200-mesh sieve and is uniform between these limits. In both the binder and top mixes the aggregates are separated into sizes at the asphalt plant and combined with the bitumen by weight and thoroughly mixed. Proper control at the asphalt plant is very essential to produce a mixture of high quality.

The material is delivered to the street in covered trucks at a minimum temperature of 250 deg.F. for the binder and 275 deg.F. for the top. In both cases it is dumped on steel pans before it is spread.

The binder is spread with hot shovels over the concrete foundation and combed to a uniform surface by hot asphalt rakes and immediately rolled with a roller weighing not less than ten tons. After the binder course has cooled sufficiently, the wearing surface course is spread in the same manner. The initial compression is with a three-wheeled roller weighing not less than 10 tons, and the final compression is carried out with a 10-ton tendem roller. When laying progresses at the rate of 200 sq.yd. or more per hour, an extra 8-ton tandem roller is required to be used.

In connection with sheet asphalt pavements the city has now developed what is called a roughened sheet asphalt surface (see Fig. 1). During construction 5 to 10 lb. per square yard of precoated trap rock chips which pass a No. 4 sieve and are retained on a No. 10 sieve are spread over the surface immediately after the initial rolling. The pavement is then given its final rolling which embeds the precoated chips in such manner as to produce a surface which has all of the non-skid qualities of the asphaltic concretes and at the same time has the dense and stable characteristics of sheet asphalt surfaces.

Contracting for Municipal Service

LOS ANGELES COUNTY, Calif., is drawing attention of governmental students and taxpayers the country over because of its contractual plan for performing governmental duties for cities which makes it possible for them to eliminate certain departments and buy the services instead. Prof. Frank M. Stewart, University of California political scientist, describes the progress of the arrangement which has been in force for nearly two years, in the January issue of *Public Management*, magazine of the International City Managers' Association.

Los Angeles County, he explains, has been authorized by the state legislation to assess property and collect taxes for all except first-class cities that elect to secure this service by ordinance. The assessor of Los Angeles County, chosen for a four-year term, assesses all real and personal property in the county and in 38 of its 44 cities as well. The county tax collector, appointed by the County Board of Supervisors under civil service regulations, collects taxes for the county, special districts, improvement districts, and for 38 cities. Los Angeles is one of the cities, paying $22,350 per year for these services. All other cities pay a small percentage of the total amount of money collected.

A civil service enabling act, enacted at the last regular session of the California legislature, authorized the legislative body of any city to adopt by ordinance a personnel system. Authority is given the legislative body of any city to contract with any other municipality or county within the state or with any state department. Under this law two cities have already signed contracts with the board of supervisors for the Los Angeles County Civil Service Commission to perform certain civil service functions for fire and police department employees.

Technology and Practices In Asphalt Paving

Experts discuss vital problems of paving mixtures, mineral aggregates, construction methods and asphalt characteristics at New Orleans meeting

ITS CHEMICAL COMPLEXITY *and the many forms in which it can be used are calling for the most intensive laboratory and field study of asphalt ever devoted to a road-building material. Most prominent in this study are the members of the Association of Asphalt Paving Technologists. At the annual convention of this organization held last month at New Orleans, La., a notable program of papers was presented by men of importance in the fields of asphalt production and road-building. To give its readers an authoritative outline of these new sources of information the following account has been prepared for* Engineering News-Record *by Prevost Hubbard, Chemical Engineer, Asphalt Institute, New York City* —EDITOR

THE Association of Asphalt Paving Technologists held its 1937 annual meeting in New Orleans on Jan. 13 and 14 in conjunction with that of the American Road Builders' Association. There were three well attended technical sessions at which sixteen papers were presented covering a wide variety of subjects on recent practices and research. The first of these sessions was opened with a motion picture illustrating the latest developments in asphalt mattress revetment on the Mississippi River. The film was furnished by the Second New Orleans District, U. S. Engineers, and its showing was accompanied by an exceedingly interesting discussion by Lieutenant George T. Derby.

The papers and discussions fell under four natural groups dealing primarily with construction methods, paving mixtures, mineral aggregates and asphalt characteristics.

Facts for constructors

Two papers giving practical determinations from experimental roads one on soil stabilization with asphalt, one on the design of thin surfaces on stabilized base and one on the practical meaning of the plastic index in stabilization offered definitely useful new data for practical road-builders.

Experiments with crack control in sheet asphalt pavements in Rhode Island were described in a paper by John V. Keily, chief, division of roads and bridges, Rhode Island Department of Public Works. The purpose of these experiments was to develop adequate means for controlling the formation of cracks in sheet asphalt produced by contraction cracks in an underlying concrete base. The paper describes the construction and behavior of numerous experimental sections of pavement in which bases of different richness and thickness were used with and without contraction joints made by insertion of thin metal strips for the full depth of the concrete.

The results so far obtained show that when the joints are installed at sufficiently frequent intervals cracks in the asphalt pavement are straight and fairly true over the plates and in no case have opened over $\frac{1}{16}$ in. Without the base joints, cracks in the asphalt pavement are ragged and indefinite and open up $\frac{1}{4}$ in. or more. In addition to the customary 1:2:5-mix 6-in. thick bases of 1:3:6-mix, 7-in. thick, were constructed. The author states "we were agreeably surprised in the results from this leaner mix, at the lack of shrinkage cracks. We feel that we have obtained a strength with the 1:3:6 mix, and 7-in. thickness equivalent to that of the 1:2:5-mix and 6-in. thickness and with the added advantage that the cost of the metal joint plates may be ultimately reduced by spreading them further apart in mixes with low shrinkage characteristics."

A. M. Miller and Earl W. Klinger, testing engineers of the State Road Commission of West Virginia presented a preliminary report of studies in the use of bitumens in soil stabilization and flexible pavement types. This paper contains the data secured by a four-year laboratory investigation from which it was found possible to develop a practical, dry-mix method for bituminous stabilization of soils. Information secured from laboratory tests on various bituminous mixtures using the Hubbard-Field stability testing apparatus made it possible to determine the best proportioning of aggregate and bitumens. A correction value for the Hubbard stability load was introduced to aid in this selection. The correct proportions of aggregates and bitumens were tested for structural strength in the Kriege loaded wheel testing assembly. Pavements of known field performance were compared with the mixtures developed in the laboratory. A formulae study which may eventually lead to better design and closer control is included in this report. The possibility that voids, density and other relations may be calculated for an unknown mixture is suggested. A detailed description of the construction of experimental test roads is given. Complete costs are presented in tabular form for sections in which asphalt cutback, asphalt emulsion, calcium chloride and retread tar were used for stabilization.

John A. Goshorn, assistant engineer, bureau of tests, Ohio Department of Highways, presented a paper on spreading low cost bituminous resurfacing mixtures. This paper described work in Ohio during the past year. The plant-mixed material was designed to use available aggregates and form a flexible but stable mat surface. Asphalt of 88-100 penetration was used in this mixture. The equipment consisted of self powered spreading and finishing pavers capable of spreading and finishing the mixture in 10-ft. widths to grade and crown without the use of supporting side forms. These pavers were of two types. One type is distinguished by the propulsion rollers riding upon the spread material and acting as rear control of the screed height. The other type paver utilized sliding straight edges which controlled the screed height. All work was done without closing the road to traffic.

The road-mix resurfacing operations required a mixing and spreading machine that would completely mix the aggregates and bituminous materials and spread the mixture in 10-ft. widths to crown and grade with one pass. The usual application consisted of from 130 to 150 lb. per sq.yd. of mixed material and the proportions were about 85 per cent coarse aggregate, 10 per cent fine aggregate and $5\frac{1}{2}$ per cent bituminous material. The liquid asphalt used in this mix was a very rapid curing pro-

duct and allowed close sequence of operations. Rolling could follow mixing and spreading very closely. Detailed description of mixes and equipment and a discussion of the leveling theory of the various pavers are given in the original paper.

Bernard E. Gray, chief highway engineer, The Asphalt Institute, presented a report on principles of design of flexible type pavements. As chairman of the newly created committee on structural design, he developed the principles underlying design of flexible type pavements, with particular reference to thickness required for different subgrade conditions. By improving the character of the subgrade through moisture control, admixtures of asphaltic products, mineral aggregates or combinations thereof, the depth of wearing course may be markedly reduced as contrasted with uncontrolled subgrades. The influence of tire area contact was discussed as well as the manner in which load is distributed to the subgrade. A formula was suggested which includes these values.

Significance of the Plastic Index in Road Stabilization was the title of a paper presented by C. A. Hogentogler, senior highway engineer, Bureau of Public Roads. This paper graphically portrayed the derivation and use of the plasticity index as determined upon soil fractions. It is expressed as the numerical difference between liquid and plastic limits of the soil. The liquid limit is the moisture content at which ten light shocks will just cause the mixture to flow and the plastic limit is the lowest percentage of moisture at which the soil can be rolled into threads ⅛ in. thick without crumbling. Generally plasticity indexes of 3 or less indicate suitability for locations subject to unusually wet conditions, 4 to 8 for average moisture conditions and 9 to 15 only for arid conditions. Indexes higher than 15 indicate unsuitability for soil road construction.

Experiences with mixtures

Joseph Zapata, assistant materials engineer, Wisconsin State Highway Commission, presented a paper entitled A Study of Bituminous Materials, Weathering Tests. He stated that bituminous mulch and mat treatments in Wisconsin had developed wide differences in behavior and service results when liquid asphaltic products meeting the same specifications were used under practically identical conditions. This was particularly true of products commonly known as slow curing. As an example, he cited two uncracked slow-curing products, one of which produced a mat that remained well bonded and the other combined with the same aggregate became brown in color and produced a loose, crumbly surface course. He stated that such physical tests as viscosity relationships, ducility measurements and indices of various kinds had failed to differentiate such products and that a study of differences in chemical composition seemed to offer the best possibilities of explaining these differences in behavior. In the investigation which he had started, it had been assumed that determination of unsaturates might serve as a measure of weather resisting characteristics of bituminous products. Such determinations were made upon original materials and the same materials after being subjected to the action of ultraviolet light. These and other tests were described in some detail and results so far obtained were shown in numerous tables and graphs. These data were presented for preliminary information only and no conclusions were drawn.

A paper which aroused considerable discussion was presented by Frank L. Raschig and P. C. Doyle of the Standard Oil Company of Ohio. This paper entitled Some Resent Research on Asphalt Pavements dealt primarily with resistance to cracking of compressed asphalt paving mixtures. Sheet asphalt was the type selected for study and the data were presented in two sections: (1) Mixtures prepared in the laboratory with asphalt cements from many different sources and (2) Sections cut from pavements, some of which under prolonged service had resisted cracking and some which had cracked badly. Characteristics of asphalt cements before mixing and after extraction were tabulated. Determinations of modulus of elasticity, modulus of rupture and total deflection of mixtures at 5, 25, and 38 deg. F., were made in general conformity with the method adopted by Rader except that the beams were tested as cantilevers. Toughness tests at these three temperatures were also reported. The findings of Rader that mixtures with relatively low modulus of elasticity and relatively high modulus of rupture best resisted cracking were confirmed.

Among the conclusions drawn by author the following are of particular interest: (1) Of the tests to which asphalts are commonly subjected all fail to indicate quality insofar as resistance to cracking is concerned except perhaps the drop in penetration during the 5-hour loss test at 325 deg.F. (2) Other things being equal, those mixtures having the highest penetration asphalt and the highest ductilities consistent with stability will prove the most resistant to cracking; (3) Of the pavements examined those which had cracked worst showed upon extraction the hardest or lowest penetration asphalt; (4) Asphalts with a high susceptibility factor should be mixed at the lowest possible temperature to prevent drop in penetration and to provide adequate film thickness and (5) In mixture design stability has been overemphasized at the expense of elasticity.

This paper was followed by a report on further research work on correlation of low temperature tests with resistance to cracking of sheet asphalt pavements, by Lloyd F. Rader, assistant professor of civil engineering, Polytechnic Institute of Brooklyn. This report describes tests on seven sections of pavement at Ann Arbor, Mich., showing various degrees of cracking from uncracked to badly cracked. The results reported support conclusions stated in the author's paper published in the 1936 proceedings of the association. In connection with conclusions drawn by Raschig and Doyle, Professor Rader stated that "in view of lack of correlation between ductility of recovered asphalt and resistance to cracking and of the discrepancies in ductilities obtained by different methods of recovery, he does not believe that a general conclusion that the condition of the pavements is poorest where the values of the ductility of the extracted asphalt are lowest can be made without a considerably greater amount of evidence being obtained to support such a statement."

Physical tests

A paper on correlation of physical tests with the service behavior of asphaltic mixtures, by Roland Vokac, The Barber Company, Inc., presented in tabular and graphic form the results of tests of ten physical properties on 106 different cold-laid asphaltic paving mixtures laid as demonstration sections on Roosevelt Boulevard, Philadelphia, in 1935. These results were compared with visual appraisals of the condition of the sections at various intervals classifying them into three general groups: (a) abrasive type, (b) normal or satisfactory and (c) shoving type. All of the tests, including voids in mix, stability, shear, compressive strength, elastic limit, modulus of elasticity, modulus of permanent deformation, index of deformation, degree of brittleness and ultimate impact resisting strength, in general showed progressive values from groups a to c. The results, however, indicated that compressive strength and elastic limit at 77 deg.F. were most critical as related to the visual appraisals. The author concludes that the compression test as described by him in a paper presented at the June, 1936 meeting of the American Society for Testing Materials correlates best with service behavior.

Some Considerations Governing the Proportioning of Bitumen in Paving Mixtures was the title of a paper by Prevost Hubbard, chemical engineer and H. C. Krieger, assistant chemist, The Asphalt Institute. The authors reviewed the two most commonly used methods of proportioning viz; the voidage method for dense-graded hot mixtures and the surface area method for low-cost cold-laid mixtures. As neither

of these methods could be practically applied to the class of mixtures covered by the other, a theory was advanced that the bitumen carrying capacity of any mineral aggregate used as a paving mixture is primarily a function of effective pore diameter of the compacted aggregate and viscosity of the bitumen. No accurate method has as yet been devised for measuring effective pore diameter. In support of the theory, however, the characteristics of certain mixtures were cited. It was shown that while in a sheet asphalt aggregate asphalt cement could be used so as to fill all but 2 per cent voids, when the fine mineral filler was eliminated, a mixture with the same asphalt bled if it contained less than 4 per cent voids. On the other hand, an extremely fine mineral powder could hold a sufficient quantity of thin dust-laying oil to fill voids to within 2 per cent and produce a stable mixture which would not bleed.

In a paper on "Physical Changes in Aggregates in Bituminous Mixtures under Compaction," M. F. Macnaughton, Milton Hersey Co., Ltd., demonstrated that degradation of the aggregate might be expected to take place both under laboratory compression of paving mixtures and their compression under traffic. Such degradation results in the formation of a larger proportion of fines or mineral dust. He concludes that under heavy traffic, or in exceptional circumstances, degradation may proceed to such an extent that aggregate void space may be materially reduced and the stability of the mixture impaired on account of excess bitumen.

R. N. Traxler, The Barber Company, Inc., presented a paper on The Evaluation of Mineral Powders as Fillers for Asphalts. Sixteen types of mineral powders and wood flour were combined in different proportions with various asphalts and their effect upon the penetration, softening point and viscosity of the asphalt was determined. Characteristics of these fillers and of mixtures produced with a given asphalt are tabulated. The author believes that the stability index of the powder is of most use in evaluating it as a filler for asphalt. The stability index is the per cent increase in viscosity resulting from an increase of one per cent by volume of filler.

R. E. Bollen, assistant testing engineer, Nebraska Department of Roads and Irrigation, submitted a paper on the selection and use of mineral fillers for low cost roads. This paper contains so much interesting and valuable detail as to make it practically impossible to summarize it in the space allotted to this review. After discussing the function of fillers in low-cost types of mixtures, the author describes the use of soil tests and stabilizing action, from the standpoint of both bitumen proportioning and characteristics of the mixture. In surface-area methods of proportioning some coefficient is assigned to the percentage of aggregate passing the 200-mesh sieve. For any particular filler the author determines this coefficient, designated the S factor, from the filler stability value. As a result of its studies, he concludes that use of a filler having an S factor of about 0.2 is highly desirable and that fillers having a high S value are too sensitive for successful use. Detailed specifications for filler are proposed.

New asphalt science

Theoretical Aspects of Asphaltic Bitumen in the Light of Recent Research was the title of a paper submitted by F. J. Nellensteyn, Director of the Government Road Laboratory, The Hague, Holland. Dr. Nellensteyn first discussed the structure of the micelle of asphaltic bitumen in the light of the modern conception that asphalt is a collodial system consisting of asphaltic bitumen micelles dispersed in an oily medium as distinguished from the older theory that it is merely a mixture of organic compounds which have no differentiated structural functions. He attributed the so-called plastic properties of asphalts to the constituents commonly known as asphaltenes and cited the blown types as showing the most marked plastic behavior as compared with straight residual products. With regard to peptizing and flocculating reagents the rule which he had developed was stated as follows: "When mixing asphaltic bitumen with liquids, which are totally miscible with the medium and protective bodies of this bitumen, flocculation occurs when the liquid has a surface-tension below 24 dynes per cm. at 25 deg. C; if the surface-tension is higher than 26 dynes per cm., total peptization takes place, while in the intermediate zone of 24-25 dynes per cm., flocculation or peptization depends upon the more or less stable character of the micelle." The properties of surface tension viscosity and adhesion of asphaltic bitumen were also discussed briefly.

A paper on production of asphalts by blending was presented by A. Holmes, process department, Standard Oil Development Co., and A. L. Raphael of the technical service division, Standard Oil Co., of New Jersey. The authors first discussed general relationships developed by blending different commercial petroleum products such as straight reduced asphalts, oxidized asphalts and fluxes. They then presented a number of tables and diagrams showing variations in characteristics of asphalts produced by blending different proportions of precipitated hard asphalt, precipitated resins and diasphalted oil obtained from propane precipitation of a reduced mid-continent petroleum. It was shown that such blending permits the production of asphalts having a wider range of properties than can be produced from the same crude by the customary methods of distillation and oxidation. The following conclusions are worthy of note: (1) By such blending it is possible to secure a lower fluidity index than by straight distillation; (2) Certain blended asphalts show an improved low temperature susceptibility over straight reduced asphalts and (3) In general, low resin content results in low susceptibility but low ductility, while high resin content produces asphalts with high susceptibility and ductility.

Adhesion Tension Measurements was the title of a paper presented by A. R. Ebberts, who described a method based upon measurement, in a manometer, of the force required to displace a liquid from the pores of a granular solid. Water, benzene and benzene solutions of three types of bitumen were the liquids used. The solids included silica sand, limestone, slag and coal. Various methods of mathematical interpretation were compared and some suggestions as to the meanings of apparent discrepancies offered. Possible reactions between the aggregates and bitumens were discussed in relation to the fact that mineral aggregates are often nonhomogeneous solids. The possibly important factor of interfacial tension between water and bitumen was considered. Due to the necessity of using solutions instead of pure bitumens, none of the values are absolute but were submitted as relatively.

Gene Abson, chief chemist, Chicago Testing Laboratory, submitted a brief discussion entitled: Notes on Recovery of Asphalts from Solution as opposed to the data and conclusions presented by Carl Bussow in a paper at the 1936 annual meeting of the association. Mr. Abson claimed that some asphalt when dissolved in benzol and kept in solution for comparatively short periods became sufficiently altered to show a distinct hardening when recovered from solution.

Water and Sewerage Facilities in Tennessee

THE biennial report of the Tennessee Department of Health for 1933-35 lists 177 public water supply systems in that state, and sewerage facilities in 93 communities; 50 of the latter were provided with some method of treatment. The following facts were classified regarding water supply: 72 received no treatment; 55 disinfection only; 3, coagulation, sedimentation and disinfection; 39, filtration followed by disinfection; and 10 iron removal. Major improvements to existing water work systems were completed in 27 towns or cities during the period covered by the report. Under the PWA program preliminary plans were approved by the State Health Department for 18 new sewerage systems; improvements to existing facilities were also approved for 7 towns. Final plans were passed for 11 towns or cities.

New Brick Paving Practices

Speakers at the National Paving Brick Association meeting in Detroit describe interesting improvements in materials and construction

EXCEPT for a paper on the use of paving brick in stadium construction, one on gas-tax diversion and one on highway safety, the three technical sessions of the National Paving Brick Association in convention at Detroit, Mich., Jan. 27-29, held closely to current paving brick research and papers on brick paving operations and base design. One day of the meeting was as usual a closed meeting of representatives of member companies, and inspection trips and committee meetings occupied another half day.

Reconstruction with brick

The first of the technical proceedings was a discussion on pavement reconstruction with brick as it has been recently carried on at Toledo, O., Ithaca, N. Y., and St. Paul, Minn.

In discussing practice at Toledo, George Schoonmaker, service director, stated that the city had rebuilt numerous streets with CWA, FERA and WPA labor, the reconstruction involving 358,000 sq.yd. of brick surface. The work had been done by turning the old brick where all conditions of surface existed at first, with all types of foundations. The foundations were repaired where possible with material originally used and where this was not possible the material used was No. 7 slag screenings compacted by 5- or 7-ton rollers. When sand and grout fillers, as first used, were found unsatisfactory, asphalt was substituted; it was poured on the surface and squeegeed, the excess filler being spaded from the surface.

The amount of new brick required varied from 5 to 50 per cent and on the total of 71 streets the average number of bricks used per square yard was 36. On the equivalent of 31 miles of 24-ft. width rebuilt, the cost to the city was $3,500 per mile with an expenditure of 5,000 man months. Filler was applied at 300 deg.F. The non-bond solution applied ahead of the filler consisted of 100 lb. calcium chloride, 3 lb. starch and 25 gal. of water. The excess filler removed by straightened mortar hoes, was returned to the kettles for reheating.

The reconstruction of 8,000 ft. of street in Ithaca, N. Y., was described by H. W. Eustance, city engineer. This WPA project costing $120,000 involved work along street railway tracks. A new concrete base was laid on widened portions, and the entire width of new base and old macadam was surfaced with an asphalt binder course 1 in. thick and on this 2 in. of rough-textured sheet asphalt. The resurfacing was with 3 in. vertical-fiber lug brick on ¾ in. mastic cushion. When dropped, the brick were painted with a separating agent, and the joints filled with asphalt. It was found that the speed of bricklaying depended on the time for filling joints and removing the excess filler. The filler was heated in 200-gal. kettles by wood fires. The removal of the old track, building 6-in. base and the brick resurfacing cost $3.75 per sq.yd.

George M. Shepard, city engineer, St. Paul, Minn., stated that with the proper base construction, proper mixing and placing of the cushion, close culling of brick, filling joints and removal of excess filler, brick pavements now being laid have excellent riding surfaces and almost indefinite life. In street widening and reconstruction in St. Paul, great care is given to obtain an accurate base contour both longitudinally and transversely. The cushion course is ¾ in. of dry sand and No. 1 heavy road oil. After applying this to the concrete it is struck off with a strike board and the brick are tested with a 10-ft. straight edge after rolling.

Laying lengthwise tried

An experiment in laying brick lengthwise of the pavement was described by Boyd V. Wright, resident engineer, Ohio Highway Department. The brick were laid on a heavy traffic street on a 7-in. reinforced concrete base 20 ft. wide with a premolded center joint.

The mastic cushion was of manufactured limestone sand and asphalt cutback, 4 to 7 per cent by weight in bituminous content. It was delivered on the job in trucks with temperature of the weather ranging from 13 to 54 deg. If the cushion material was below 60 deg. when delivered on the job it could not be spread. The bricks used were vertical lug bricks, 3 x 4 x 8½ in., laid longitudinally 3 in. deep on a ¾-in. mastic cushion.

The Ohio highway department set up supplemental specifications for handling and setting the brick. On tangents the alignment was not to deviate more than 1½ in. in 20 ft., and individual bricks were not to vary from normal sizes by more than ⅛ in. in width or ½ in. in length. The laying face was diagonal or at an angle of 45 deg. with the curb.

Trouble was encountered in keeping the courses straight. Templets of 1-in. lumber were drawn from curb to curb and finishing nails in the bottom of the templets marked the bed off in lanes by indents into the cushion. The templets were used to line up the brick at every sixth row; spacing was confined to the lanes.

Conclusions drawn from the experiment included: Bricks should never be spaced or shifted; once laid they should never be disturbed except by the roller or by lifting vertically; laying should start at the curb and when courses do not come out even, the remaining space should be filled with bats; it is impracticable to try to get straighter courses and still maintain the ultimate in fine riding qualities.

In discussion V. N. Holderman, superintendent, Beightler & Hussey, contractors, stated that experience on four jobs on which 4,000,000 brick were laid longitudinally indicated that there was but little difference in cost as compared to laying in transverse rows; that exact dimensions of brick would eliminate any difficulty in laying them in longitudinal rows. On one particular job referred to, a space of about 3 in. along the edge of the outside longitudinal course necessitated the distribution of space between the rows with a tendency to cause an irregular surface, and to require additional labor in laying, and an additional filler requirement of about 2 lb. per sq. yd. of surface.

The experience on these jobs further indicated that more droppers could be used in laying longitudinal courses. A conveyor used on the first job was soon abandoned in favor of man power and tongs. The cost of paving operation was reduced about 2c per sq.yd. by longitudinal paving, with smoother riding and 25 per cent less noise in the car than when brick were laid transversely.

Following this discussion, Boyd V. Wright referred to the difference of opinion on the advantages of longitudinal and transverse laying of paving courses, and expressed the belief that uniformity in size and shape would make paving brick a greatly improved road material.

A comprehensive report by the association research bureau gave results at the end of one year of the behavior of the various joint fillers in the Lancaster,

O., test road. The report was made by H. Z. Schofield, director of the bureau. The research, he said, was concentrated on 21 joint fillers for brick surface.

The fillers tested included a 1 to 4 mixture of cement and sand swept into the joints; a mix of one part of cement, one part of special emulsion and three parts sand; plasticized sulphur and asphaltic mixtures; mineral fill asphalts; blended asphalt without mineral filler; mastic asphalt; asphalt F-1 from different crude bases and of different melting points; special pitch with 5 per cent asbestos; pitch; hard pitch; asphalt with 38 per cent stale flour, and other special pitches.

Based on the one-year tests it was found in general that 1 part cement to 4 parts sand swept into the joints; 1 part cement and 1 part special emulsion to 3 sand; asphalt of low melting point; and the special pitch with 5 per cent asbestos were entirely unsatisfactory. The cement-sand filler showed evidence of cracking, and in the case of the cement-sand mixture with special emulsion, much of the filler was gone leaving loose bricks. Asphalt of low melting point was found to have worked from the center of the crown to the edges of the road. The special pitch with 5 per cent asbestos proved too soft and sticky, receding in the joints and flowing out from the crown. Mastic asphalt with 40 per cent sand proved hard to mix uniformly. Asphalts F-1 were found to exude oil and with the hard pitches, heavy exuding was found with resultant chipping of the bricks.

Type B, plasticized sulphur-asphalt filler, of 60 per cent asphalt and 40 per cent sulphur, proved very satisfactory, as did also the mineral fill asphalt, the blended asphalt without mineral filler and one special pitch which showed no receding or exuding. No attempts have been made to find definite results after one year but the four satisfactory types of filler are to be used further.

Current research by the bureau also includes the use of thermocouples to get the temperatures at various points in and below the road and road bed, including points in the brick and fillers. The minimum temperature of subgrade 30 in. below the road bed has been found to be 28 deg.F. and the maximum was 82 deg.F.

Large slabs 5 ft. long, 2½ ft. wide and 4 in. thick, of brick joined by ordinary masonry cement were tested with a view to determining the possibility of using large slabs made up in the factory to be laid on the road bed. With ends supported, these slabs supported loads up to 5,000 lb.

F. H. Jackson, senior engineer of tests, Bureau of Public Roads, in discussing the quality of the paving brick unit, suggested tests to determine the chipping and cracking qualities, information not entirely given by the rattler test. A possible correlation between the chipping quality and the waste after the rattler test was suggested. R. R. Litehiser, chief engineer of tests, Ohio Highway Department, referred to the large number of cracked bricks encountered at present and suggested tests for cracking and also care in manufacturing which would help eliminate defective edges which add to the tendency to crack.

P. D. Gephart, H. H. Robertson Co., referring to the claim that mid-continent asphalts change very little over a wide temperature range while other types have other good qualities, discussed efforts to blend the different types to get a combination having the best qualities of all.

G. E. Martin, The Barrett Co., stated that after much experimentation on tar mastic cushion beds, his efforts to mix pitch with other substances had been abandoned and efforts centered on improving the pitch itself with the result that what is believed to be a very satisfactory product has been developed.

W. W. Duecker, Mellon Institute, in experiments on sulphur, found it difficult to eliminate odor when heating filler for the laying process. Filler type B, listed by Mr. Schofield, 60 per cent asphalt and 40 per cent sulphur, is one that is being developed for the market. It gives off a little hydrogen sulphite in laying, but the odor is not bad and stops in 24 hours. Asphalts were referred to as colloids, likely to disintegrate and fall apart under excessive vibration.

P. E. Masheter, Ohio Highway Department, outlined briefly Ohio's experience in using portland cement concrete base. Early roads were laid on the lightest possible bases, and faulty bases are considered a principal cause of surface failures. In 1927, Ohio used 1:6 concrete 5½ in. thick. In 1930, the specification was 1:7½ concrete 6 in. thick, which was satisfactory on roads with good subgrade and light traffic. Cushion beds seep up between the bricks and spread over the surface. Longitudinal joints have been used and found worth while and transverse joints have been used with varied success. Infiltration of particles into contracted joints added new strains on expansion. Joints are spaced up to 100 ft. Joint fillers cause the bricks to hump upon expansion of the base. In 1936 the trend has been to 7-in. reinforced-concrete base which showed improvement over plain concrete base. Better contraction joints were used. In 1937, concrete will be 1:7½, 7 in. thick, reinforced with longitudinal joints but no transverse joints. Reinforcing will distribute the small cracks that form.

A new road is to be built of brick settled into concrete by vibrating machines. The conclusion was drawn that failures in the base will cause surface failures; that reinforcement ends the loss through cushion cracks; that the use of longitudinal joints is helpful.

C. D. Willman, speaking for V. B. Steinbaugh, Michigan Highway Department, stated that very little brick is used in Michigan. Just recently it has been found very satisfactory. In road building, the soil, the temperature and the use of the pavement must be considered. Bricks are used only where extra heavy traffic occurs. Reinforced concrete base courses are finished smoothly with longitudinal, but no transverse joints. Mastic cushion is used. Transverse planes of weakness, provided at intervals, do not crack far enough to cause trouble because the reinforcing steel ties them together. Otherwise there are no expansion joints. Existing concrete base is sometimes surfaced with a thin mortar bed for smoothness before applying the mastic coat and brick.

General papers

Murray D. Van Wagoner, State Highway Commissioner of Michigan, spoke on "Diversion of Highway Funds."

Mr. Van Wagoner referred to the seriousness of such diversion and the difficulty in stopping a practice which originated during the depression when state revenues were greatly reduced. Diversion was stated to be a barrier against new construction and maintenance and to prevent elimination of dangerous curves and intersections. The speaker suggested a national amendment to federal-aid law prohibiting diversion and recommend a national clearing house to coordinate activities in the various states on diversion problems, in a permanent well planned effort rather than by spasmodic action.

Remarks by the chairman on the use of paving brick in Europe were illustrated by slides. In Holland nearly all roads are brick on concrete base. Brick paving finds little use in France and Germany, but many brick roads exist in Czechoslovakia. On most of the European roads the brick are laid in herringbone fashion. Italian bricks are very fine, more than 50 per cent glass; and Holland bricks are made by the plastic press method. Little is known about glass in bricks and further studies are being made. It is known that the type of glass affects the strength.

J. Davis Wilson, New Philadelphia, Ohio, prefaced a talk on the use of paving brick in stadium construction with a brief resume of some of the earliest known uses of brick in paving construction and referred to one of the first paving-brick streets, constructed in 1880 in Charleston, N. C. using bricks somewhat wedge shaped, 2½ in. wide on top, 3½ in. wide on the bottom, 4 in. high and 9 in. long.

Stadium construction was referred to as an outlet for odd-size paving brick. The particular stadium described has a capacity of 4,000 and was constructed at Dover, Ohio, as a WPA project designed to use a maximum hand labor.

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Beyond the City Limits

FROM TIME TO TIME events and conditions throw light on the influences that a city exercises over the surrounding region. Tax difficulties as well as traffic and service problems give constant proof that the suburbs feed on the city and the city on its suburbs. In other form, their interdependence is reflected in the fact that some large cities operate water or sewerage service well beyond the city limits. Control of suburban subdivision by cities has also been discussed—an outgrowth of the reaction of unrestrained suburban land-use operations on the welfare of the city. New recognition of this interrelation between city and region appears in Arthur E. Morgan's recent demand that when a city undertakes to operate its electric service it should also take over the service of the surrounding country area—on the ground that in most cases this area can not obtain efficient service independently. This demand, which duplicates a suggestion which he made to a civic group a few months ago, challenges the attention of municipal officials and of all who are interested in efficient public organization. It points to the fact that there are vital city interests beyond the city limits, and that ways need to be found to safeguard these interests.

Freezing in Construction

FREEZING A SOIL BANK to check slides into a cofferdam, as was done at Grand Coulee Dam, is noteworthy not because of the novelty or form of the treatment but because of the infrequency of its application to construction operations in this country. A few decades ago the freezing process as an aid to shaft sinking was tried in this country, mostly with indifferent success, due chiefly to the slowness of the process and uncertainty as to when a wall of earth had been frozen to sufficient thickness to permit excavation to go forward within the shaft. Abroad the process has been used extensively and still is. A notable example is found in the Antwerp shaft of the Schelde River vehicle tunnel, an undertaking in which American engineers participated. Without doubt, the process has distinct advantages for large shafts due to the elimination of bracing, but the long period required for freezing—four months for the Antwerp shaft—is a serious discouragement, and there are many unknowns. It is in respect to eliminating unknowns that the Grand Coulee work is specially significant. The careful study of the freezing process that was made there, as reported in this issue, should help to make the process of greater service to construction.

Toward Control

GRATIFYING PROGRESS toward mastering the silicosis plague was accomplished through the work of the technical committees of the Silicosis Conference that reported last week in a session at the Department of Labor at Washington, after months of hard work. Two items stand out in the accomplishment of the committees: a clear formulation of the known facts of the disease and its medical and engineering control, and a well-considered statement of the principles that must underlie regulatory and compensation procedure. The studies explored each of the numerous aspects of sound and socially attainable regulation, including the problem of employing workers in silica-dust occupations, elimination of the tuberculous, protection of affected persons, supervision of hazardous occupations, and finally insurance. As was to be expected, health examinations of workers at time of employment and afterward were declared essential, and while the conference revealed that labor representatives are opposed to such examinations their necessity in the worker's own interest is too obvious to allow such objections to prevail. Much more remains to be learned about silicosis, it is apparent—more than is yet known, indeed. Especially in construction operations there is an appalling amount of ignorance on the occurrence and magnitude of the silicosis hazard. Therefore the work of the conference, if its effects are to be real and lasting, should be carried forward by a continuing study.

Fundamental Exploration

WITH THE PUBLICATION of the countrywide study of drainage-basin development projects that was carried out last year by the Wolman committee for the National Resources Committee there becomes available a reconnaisance of essential river-improvement projects that will long be of fundamental value to the country. The excellence of the study is appreciated by all who had opportunity to observe the care and ability with which the work was carried out. On this foundation it will be possible to build a more complete study, leading up to a tentative program of urgent works. It is important to remember that the report presents only a hurried reconnaisance, and by no means an exhaustive, carefully studied engineering finding. The work consisted largely of mere compilation of projects that had previously been advanced and partly developed by various interests. Manifestly, only superficial scrutiny could be given to these projects by the compiling organization, for thousands of major undertakings had to be considered and the entire work had to be carried through in less than six months. The resulting inventory is at best a preliminary list of projects whose individual merits and whose relation to each other will require intensive study; it is in no sense a list of projects ready for immediate construction. But as the first fundamental outline of useful and needed river-improvement projects ever brought together it is a valuable starting-point for further studies.

Economic Dictatorship

IN A REPORT PUBLISHED together with the drainage-basin study, the National Resources Committee makes revolutionary proposals on public-works construction. Apparently the committee would centralize all public-works thinking, all initiative and all financing in Washington—a virtual economic dictatorship. First of all there would be a planning board to plan projects and set up programs, for Congress to approve. Once such a list were approved, Congress would merely appropriate a lump sum of money for works construction, but the decision to build this or that project would be made by an executive board. The central planning board would also approve programs for local public works, and after

the federal government had made a "thorough inquiry" into state and local finances there would be federal money allocations or grants to aid in the construction of local projects, in the discretion of a cost-allocating board at Washington. Thus Uncle Sam would be made the angel of city and state works, and the whole mechanism of public-improvement activities in the country would be directed by two or three small groups in Washington. Incidentals to the scheme would be "the loaning of technicians and scientific workers by the federal government to state and local agencies (and) loans of machinery and tools." To envision the future of our development under such a system is difficult. Fortunately there is little likelihood that the committee's recommendations will find favor with Congress or the people. In fact, the report may not be read by more than half a dozen people all told, for a 200,000-word document is an effective burying-ground for ideas, good or bad.

Mississippi and Ohio

AFTER an unexampled course of destruction along the Ohio, the flood that is now passing down the Mississippi seems to have spent its fury. Its attack on the levees from Cairo to Memphis has been beaten off, and the floodwave's remaining course to the sea threatens no major alarms.

Saving the ever-present chance of a surprise, then, the great 1937 winter flood is passing into history. Its effects cannot be appraised fully at this early date, but one or two leading facts are clear even now.

MOST important of all, the flood-control works have withstood their first test—a test of phenomenal severity. From Cairo down, the long, thin front line of defenses is holding against a great flood for the first time since white men entered the valley.

The levees are proving adequate to hold a flood much greater than that of 1927. The Birds Point floodway was effective; it saved Cairo from destruction. Gen. Ferguson's wonderful series of cutoffs below Arkansas City, though still waiting a full check of results when the water goes down, is at present surpassing highest expectations; lowering the flood height by many feet, the cutoffs are keeping the river out of the Boeuf basin. The Atchafalaya and Bonnet Carré appear safe.

Thus the flood drama sketched in advance in this column two weeks ago has passed its crisis. The overshadowing question whether the flood-control works could survive the "superflood" for which they were designed is answered affirmatively. The present flood closely approached superflood dimensions in the reach just below Cairo, and yet with the scanty design freeboard of only 1 ft. the unseasoned levees held. Gagings indicate that the floodway was able to carry even more than the 400,000 sec.-ft. on which the engineers counted.

BUT NOW ARISES the vital question whether the valley is permanently secure.

The reader will find it instructive to follow Charles S. Hill's review of the origin and main elements of the 1928 flood-control project as published on another page. From that review he will see that the project capacity in the Memphis reach is about 2¼ million cubic feet per second. That is the project "superflood" volume, supposed to represent the greatest concentration of waters that can ever occur, even under the most extraordinary combination of flood discharges from all the streams above, the Missouri and the upper Mississippi as well as the Ohio. By building a channel to carry such a possible but supposedly quite improbable flow, the army engineers believed that they were protecting the valley for all time to come.

Against this, consider the facts of today. The present flood, coming from the Ohio alone, reached the superflood volume. What disaster might we be witnessing had the Missouri and the upper Mississippi also sent down a substantial flood? In fact, even without high water in these two great streams, only a slight shift in the position of the storm might have produced a sharper and higher flood crest.

THUS, the record of the flood clearly demonstrates that the Jadwin plan is inadequate. Its assumed superflood is too small for security. Even as we write —for the flood is not yet over—the safety of the valley hangs on the chance of rains on the tributaries. If it is to be safe against all flood hazard now and in future, a new and much greater plan, able to resist the maximum possible flood, must be created. The army engineers have not ended their task; they have only begun.

The fortnight's experiences show also that the bitterly fought "fuseplug" levees are a menace that cannot be tolerated. Fortunately they have already been abandoned, for under the Markham revision of the project Eudora and Morganza are to have controlled spillways. Birds Point is equally entitled to a controlled spillway, however. And simultaneous with this change the whole system of floodway occupancy needs to be reviewed. The tense situation of two weeks ago, when the Birds Pointers had to be driven off their lands by force at the time the fuseplug was dynamited, ought never be repeated!

IN THE OHIO VALLEY, general devastation stands out in stark contrast with the Mississippi's security. To make the Ohio safe is the next great undertaking. Many years will elapse, however, before detention reservoirs can cut down the Ohio's crests even moderately, and until then protection must be provided by other means.

For safety against destruction like that of the past two weeks the river flats, the high-water benches and the flood-channel slopes that have been occupied by cities and towns must be cleared. It is inexcusable folly to permit habitation of flood channels or to tolerate obstruction of flood-flow areas by human works. And if anything it is even more inexcusable to place the services that mean the life of thousands—waterworks, power stations, telephone exchanges and gas plants—where they can be paralyzed by flood.

THERE IS WORK AHEAD, a vast volume of work, to bring full flood safety to the valleys of the interior basin. That the nation through its federal government will take over a large share of the work cannot be doubted; in planning and building the storage basins and retarding dams which in due time will reduce the peak violence of floods, the national government is certain to take a leading part. But making the valley's flood zone less vulnerable is within the power of the states and cities concerned, and it is their responsibility alone. The immediate task set by the valley's greatest flood, therefore, is a task that the valley itself must undertake.

CURRENT NEWS

Mississippi Levee System Holding Flood within Bounds

Army engineers regard fight to hold levees intact as won as the crest of the flood moves downstream from Cairo, though high winds and waves batter sandbag topping—Cutoffs below Arkansas City are lowering the flood heights well below top of levees and no trouble is expected in Vicksburg and New Orleans districts.

By H. W. Richardson
Associate Editor, Engineering News-Record

THE LEVEE system of the Mississippi River is holding without a break as the crest of the flood moves upon Memphis from Cairo and other points upstream. Barring sudden heavy rainfall in the tributary areas above Memphis, the army engineers regard their fight to hold the superflood within the levee lines as won. For nearly two weeks they have directed a force of 120,000 workmen in raising low spots in the levee to safe levels, in fighting to keep seawalls at Cairo and Hickman from being overstopped, and in checking threatening sandboils and sloughs in nearly 600 miles of levee. So far the engineers have been victorious in their fight to hold the river in check at all points except the dike at Bessie Landing, opposite and upstream from New Madrid. Here the river broke through the old dike, built to direct the current around New Madrid bend, in spite of strenuous efforts on the part of a large army of workmen. There is a possibility that the river may cut through a new channel across the neck, but so far the cutoff current has been largely stilled by backwater, and apparently no serious cutting has taken place below original ground level.

A high wind over the past week-end whipped the swollen river into high waves that dashed against sandbag topping hurriedly placed at many vulnerable points, but no breaks have occurred as yet. Sandboils are causing some concern, and danger from these seeps will increase with the duration of the crest. Treatment of boils is by simple ringing with sandbags in order to build up a back head of water sufficient, with friction through the soil, to overcome the flood head. Near Dundee, Miss., several large boils were checked by building up a high ring of bags, then pumping the enclosure full of water before the seepage became serious.

As this is being written the crest is passing Memphis at a stage slightly in excess of 50 ft. on the engineer's gage (about 1.6 ft. higher than the weather bureau gage). At Cairo the water has dropped about 7 in., though the drop is very slow, only about 0.01 ft. per hr. This slow decline indicates that the flood will drag out for many days as it moves down the river. Meanwhile, there is always the danger of heavy rains on the tributaries, especially the Arkansas and Red Rivers, which might cause trouble.

Floods Under Control in Vicksburg and New Orleans Districts

Vicksburg, Miss., Feb. 5, 1937.

The army engineers of the Vicksburg and New Orleans districts comprising the lower Mississippi Valley are calmly awaiting the high waters rolling down from Cairo, confident that no trouble will be experienced in confining the flood within the levee lines. The greatest safeguard against excessive flood heights in these districts is the twelve cutoffs opened since the last big flood. They are materially reducing crest levels to well below top of levees. The enlarged and raised levee sections and the Bonnet Carré Spillway are additional factors of safety.

The remarkable effect of the cutoffs in reducing flood heights is shown by comparison of present gage readings at Arkansas City, below the mouth of the Arkansas, with those of previous floods. Prior to the present flood the greatest amount of water ever carried between the levees at this point was 1,700,000 sec.-ft. in 1929 when the crest registered 58.9 on the Arkansas City gage. The same flow was reached in the current flood on Jan. 31 but the gage then read only 46, or 12.9 ft. lower than the corresponding stage in 1929. As the river has been slowly rising since that date the flow is now the greatest ever carried between levees through that area.

From the water now in sight upstream, the peak flow expected in the present flood is 2,100,000 sec.-ft. at Arkansas City. This discharge, 25 per cent greater than the 1929 peak flow, will raise the river to only 54.5 ft., still 4.4 ft. below the 1929 peak stage. Lowering effect of the cutoffs is greatest at Arkansas City which is a few miles upstream from the cutoffs of the three Greenville bends, the upper cutoffs in the system. Below Greenville the effect is not so pronounced, but from there down river the levees are built to 1928 and modified 1931 grade and section so there is no fear of trouble. Just below Greenville the expected crest line follows closely that of 1929 flood for some distance, then rises slowly above the previous height to a maximum of 4 ft. above the previous height at Vicksburg. The crest is not expected at Vicksburg until about Feb. 14.

The twelve cutoffs, seven of which are in the Vicksburg district and five in the New Orleans area, shorten the river by 100 miles. The present current through the cutoff areas is estimated as high as 12 m.p.h., and many river boats are forced to take the old bends around the cuts in going upstream.

Low levees being raised

The levees on the Mississippi side are in excellent shape—built to new section. From Yancopin, Ark. (on the Arkansas River almost due north of Arkansas City on the map, p. 223) the main levee that follows the south bank of the Arkansas and then the west bank of the Mississippi is only at 1914 grade for 62 miles. This section was to act as fuseplug for the Boeuf floodway. From Walker Bend (the first bend below Greenville) southward the west bank levee is at modified 1931 grade. In the 62 miles of fuseplug embankment are several sections totaling three miles that are slightly below 1914 grade. These spots are being raised by dragline, truck and bulldozers, no hand work being done in contrast to similar work in the Memphis district. A few spur dikes are being strengthened to prevent possible cutting of main embankments. The fuseplug will not overflow. Below Vicksburg the only preparations for the flood consist of closing drainage and railroad gaps through the main levees. The Bonnet Carré Spillway is in action, though on Feb. 3 it was discharging only 75,000 sec.-ft. with 6 in. head over the 18-ft. sill and 30 in. over the 16-ft. sill.

The evacuation order

The chief worry of army engineers in the lower valley is the quieting of residents scared and stirred up by the absurd order of Secretary Woodring to evacuate the whole valley for 50 miles both sides of the river. Except for the lower areas covered by backwaters, which occurs with even much lesser floods, the lower valley is dry and expects to remain that way. Only excessive rainfall in the Arkansas basin in the next few days can change the present outlook.

The above summary of conditions in the lower valley was written from personal observations made on a trip from Vicksburg to Arkansas City and return by car and by boat, with Lt. Col. L. E. Oliver, district engineer, Vicksburg, and Captains Karl Schilling and J. S. Seybold, of the Vicksburg office. We inspected the levee line along the west bank of the river, looked over the levee raising projects and spur dike revetment work. Today we returned to Vicksburg via boat, coming down through all of the seven cutoffs between here and Arkansas City. Data on flood flow and gage heights was obtained from profiles prepared by Col. Oliver's staff. Some of these profiles will appear in next week's issue.

TVA Explains Its Position In Reservoir Storage Release

The Tennessee Valley Authority is releasing about 40,000 sec.-ft. from storage in Joe Wheeler and Norris reservoirs in preparation for possible additional floods in the near future. Col. R. G. Powell, U. S. division engineer, Cincinnati, asked the TVA to withhold the release until the floods in the lower Ohio and Mississippi valleys had dropped to safe levels. In explaining its position, the Authority wired Col. Powell as follows:

"Release at Wheeler had already been reduced before receipt of your telegram of Feb. 6 and is being held to the minimum practicable. Norris and Wheeler reservoirs were filled until great damage to highways, railways and towns was imminent, in order to give maximum reduction of flood crest at Paducah. It is necessary now to release water as rapidly as safe so as to be able to handle subsequent flood flows. We estimate water released from Wheeler reaches Paducah in about four days."

Word from Carl A. Bock, assistant chief engineer, TVA, indicates Norris reservoir was at el. 1030.6 before release began. It requires ten days for flow from Norris to reach Paducah. At present Norris reservoir has only a moderate storage available for local flood control.

Flood Damage to Roads Near Twenty-five Million

Officials in Washington are thinking of highway flood damage in terms of some $25,000,000. Of that total they roughly estimate that rehabilitation of federal aid roads is likely to call for expenditures of $12,000,000 to $15,000,000. It is admitted that no accurate estimates can be made until more is known as to what has happened to bridges.

Local estimates run to a higher total but it is believed these will be revised downward. Reports from Ohio indicate that 750 mi. of state roads are inundated, in addition to the federal aid roads. That damage alone is estimated preliminarily by state officials as $10,000,000.

Representative Robert T. Secrest of Ohio has introduced a joint resolution authorizing an appropriation of $100,000,000 for the repair of roads damaged by floods in the entire overflowed area, to be allocated by the President.

LONGEST CONCRETE RIGID FRAME COMPLETED AT SEATTLE

THE LONGEST rigid frame bridge of reinforced concrete yet to be constructed in this country is claimed by Seattle, where the Schmitz Park bridge, shown above, has recently been completed. This structure has a 175-ft. span, center to center of hinges, carries a 54-ft. roadway, two 6-ft. sidewalks, and concrete railings, and is designed for H-20 loading. Its four ribs are of cellular type (*ENR* Nov. 5, 1936, p. 637). Other long concrete rigid frames in this country include the Martinez Street bridge, San Antonio, 101.5 ft.; the Loup River Canal bridge, 109 ft.; and the Lincoln Park bridge at Kenosha, Wis., 146 ft.

Big Housing Scheme In New Zealand

Five thousand houses per year to be built and owned by the government and rented to tenants

An intense housing shortage in New Zealand has led to the adoption of a large program of government financed housing construction. The houses will be built by the state with its own funds, without any bond issue, and will be rented to tenants, the rentals accruing to the government. Considerable variation in type for the houses has been secured; about 65 ground plans have been prepared, each of which can be varied about ten times as to elevation, and with varying color scheme. About 60 per cent of the houses will be built of brick veneer on wooden frames with plaster cast wallboard, tile roof, and concrete foundation; the remaining 40 per cent will be of wood frame construction.

It is planned to build a minimum of about 5,000 houses a year, but the real limiting factor on the size of the program is the amount of labor available. The construction of 40 per cent of the houses of wood frame arises directly from the fact that there is a shortage of bricklayers, and when this shortage has been overcome it is planned to build all the houses of brick veneer. The great shortage of all types of building workers in New Zealand has led a considerable number of Australian contractors to express intention of bidding on the work. The housing ministry hopes for some American bids, and states, for the information of such bidders, that in the past houses of this type have cost from $2,800 to $4,000. The houses will be built by contract under a scheme which has some rather unusual features. Contractors will be asked to insert in their contracts unit prices for certain types of work and lines of materials. If the state, with its mass buying power, can secure any line of material at a cheaper rate than that quoted by the contractor, the contract will be adjusted accordingly. In this way, it is hoped that individual contractors will receive the full benefits of the state's buying power. A quantity schedule for each house has been taken off by the ministry for housing, so that each individual contractor will not have to take off a quantity schedule in each case.

Contracts in the larger cities will be for about 400 houses, and contractors will be allowed to bid in groups of not less than twenty. The lowest bid for each twenty will be accepted, although one large firm can obtain as many as 400.

OHIO AND MISSISSIPPI STAGES AND DISCHARGES FROM U. S. GEOLOGICAL SURVEY

Location	Date	1937 Peak Gage Height	1937 Peak Discharge Sec.-Ft.	Previously Recorded Maximum Stages Year	Previously Recorded Maximum Stages Gage Height
Ohio River at:					
Pittsburgh, Pa.	Jan. 26	34.5	305,000	1936	46.0
Wheeling, W. Va.	Jan. 26	48.5	382,000	1936	55.5
Huntington, W. Va.	Jan. 27	67.7	642,000	1913	64.5
Cincinnati, Ohio.	Jan. 26	80.0	950,000	1884	71.1
Louisville, Ky.	Jan. 27	57.1	1,100,000	1884	46.7
Evansville, Ind.	Jan. 31	53.7	.....	1913	48.4
Paducah, Ky.	Feb. 3	60.75	1,850,000*	1913	54.3
Cairo, Ill.	Feb. 4	59.6	.....	1927	56.4
Mississippi River at:					
St. Louis, Mo.	Feb. 5	12.2	172,000	1844	41.39
Cape Girardeau, Mo.	Feb. 5	27.2	183,000	....	....
Memphis, Tenn.	Feb. 6	48.0	1,910,000	1913	45.2
Vicksburg, Miss.	Feb. 5	48.3	1,810,000	1927	58.4
Atchafalaya River at:					
Krotz Springs, La.	Feb. 6	33.5	320,000	....	....

Note: Records at Mississippi River and Atchafalaya River stations show the latest available information and may not represent peaks.

* Revised.

Resources Committee Submits National Public Works Program

President Roosevelt recommends adoption of a standing six-year public works plan, based on selection and priority of projects. Planning and allocation of funds would be placed under direction of a central advisory board.

In a special message to Congress on Feb. 3, President Roosevelt urged the adoption of long-range public-works planning and construction. He referred specifically to a three-part report submitted by the National Resources Committee, the first section of which recommends the establishment of a standing six-year program looking toward comprehensive development of public works. The second part of the report, dealing with water resources, outlines specific proposals regarding drainage basin problems and programs; the final section lays down suggested procedures concerning the division of costs on public works and their planning.

To carry out the scheme of integrated planning the committee proposes the establishment of a permanent central planning organization to formulate and annually revise a six-year program of federal construction. A list of projects would then be submitted to Congress for approval. To cover expenditures on approved projects a lump sum annual appropriation is to be provided in the regular budget. These funds would then be allocated to appropriate construction agencies at the discretion of the central planning board.

Stating that numerous bureaus and departments at all levels of the government are engaged in planning public works and that the multiplicity of the projects and agencies proposing them caused confusion, the committee pointed out the necessity for an advisory staff that would integrate planning on a broader scale, so that all parts can be seen in their true relationship to the whole. This would call for coordination of existing local, state, regional and national planning bodies and an extension of their activities.

Financing the work

It is proposed that the authority of a federal planning board engaged with the development of a public-works program should include not only the listing of the physical and engineering features of each approved project but also recommendations as to division of its costs. Impartial boards or panels of assessors or appraisers might be set up in some instances, and authorized either to decide the ratio of financial burden for the localities, or at least make recommendations.

It is held that former systems of arbitrary apportionment cannot be applied to long time planning of works or to employment stabilization without modification in procedure and substance. The committee stated that a different apportionment will be necessary in periods of depression from that used at other times. It therefore appears necessary, says the committee, to place these determinations of cost divisions in the hands of administrative agencies who will weigh and balance all the factors.

Inasmuch as the ability of subordinate governments to assume financial obligation for public works is inevitably connected with any divisions of costs that are developed, it is felt that plans for the future should include not only a thorough inquiry into federal, state and local financing but an attempt to correct these conditions which have prevented subordinate governments from carrying a fair portion of the load during the past depression. The problems of finance and of intergovernmental relations can thus be brought into clear relief.

The report states that so important is the complete coordination of fiscal policy and public works financing and planning with the monetary and the credit policy of the nation that it proposes that final review and recommendations be made by a fiscal advisory committee appointed by the President.

The adoption of six-year programs by cities and states was urged by the committee, in order that local government projects could be tied-in with federal works. This would be particularly advantageous during periods of depression when proper timing of work would assist economic stabilization.

Drainage basin studies

The second part of the report deals with a nationwide investigation of drainage basins undertaken with a view toward determining the principal water problems in various areas and to present an integrated pattern of development and control designed to solve these problems. Specific investigations of construction projects, with priority of importance and time, were recommended for the 127 basins that were studied by 14 regional consultants. Some of the projects are federal, others state, and still others local.

The Water Resources Committee found it impracticable and undesirable to assign absolute priority to projects for the country as a whole, for regional groups of drainage basins, or even for individual basins. It was feasible, however, for individual basins and for groups of basins, to make a classification of projects in three broad groups, a procedure deemed adequate for all practical purposes. The groups adopted were as follows.

Group A (immediate)—projects which are ready for construction or study and which should be undertaken as soon as possible.

Group B (deferred)—projects which, while desirable for immediate construction or study, are obstructed for the present by legal, administrative or other difficulties. Group B also includes projects which would follow Group A in sequence of construction or of study.

Group C (indeterminate)—projects which, although included in the plan of development for the basin, should follow Group B in sequence of construction or whose specific priority in the program is as yet indeterminate.

Tables and maps accompanying the report outline desirable projects for further investigation and projects which are ready for construction; also listed were those undertakings which are now under way and recommended for completion.

Estimated cost of the projects for investigation in Group A is about $20,000,000; construction items in Group A total $1,727,000,000. Deferred construction projects, falling into Group B, total $1,024,000,000.

It was stated that the project lists embodied in the report provide a reservoir of meritorious undertakings intended to cover a period of ten or more years, but the program is readily adjustable to the requirements of any policy that may be adopted with respect to either the rate or the mode of public expenditures, whether federal or non-federal, on water resources.

Division of costs

The final section of the report is an academic study of division of cost and responsibliity for different types of public works among the federal, state, local and private interests concerned with these projects. It summarizes research material gathered in the course of an exploratory study in special phases of public works programming by a group of economists and financial advisors. The various papers included in this section were submitted as supporting data but the National Resources Committee stated that it assumes no responsibility for the views and opinions of the authors.

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Housing Projects Redesigned In Minneapolis and Evansville

Redesign of Summer Fields Homes, the $3,500,000 PWA housing project in Minneapolis has been authorized. Bids for construction of superstructures on this project were rejected last November because they exceeded the estimate and would have raised the apartment rentals too high. Bids for construction of superstructures will be advertised as soon as the revised plans have been prepared.

Aiming to reduce the construction cost per room and at the same time slightly increase the total annual revenue from the project, the new plans will call for 472 units instead of the original 451, bringing the total number of rooms to 1,725. This will involve the construction of four additional two-story row houses. In its revised form, the project will consist of 46 two-story row houses and flats and four three-story apartment houses, plus three small heating-unit structures and one maintenance and repair building.

Specifications for the buildings will also be altered to reduce the cost of construction without reducing substantially the dwelling facilities. All structures will be of brick or brick and tile, the site plan will be somewhat simplified, and heat will be supplied from five separate units. All of the foundations already completed will be used for the redesigned superstructure.

Authorization has also been made for revision of the plans and specifications for Lincoln Gardens, a million dollar project in Evansville, Ind. Bids based on the revised plans are to be called for early in April. Redesign of this project also followed rejection of the bids originally received on the grounds that they were too high.

The new plan calls for 191 dwelling units, 53 more than called for originally.

Tunnel Contract Awarded

Mason & Hanger Company, the only bidder, will build the second tube of the Mid-Town Hudson Tunnel

The Port of New York Authority has awarded the contract for construction of the second tube of the Mid-Town Hudson Tunnel, from West 39th St. in Manhattan to Weehawken, N. J., to the Mason & Hanger Company on its bid of $8,764,000. The Mason & Hanger Co., which is now building the first tube of the tunnel, was the only bidder for the work, and its bid was below the engineer's estimate for the project. The accepted bid was based on the use of structural steel lining in the rock tunnel portions of the north tube. An alternate bid of $8,939,800 was made covering the use of cast iron lining. The contract for the cast iron rings that will be used to line the under-water portion of the tunnel was awarded by the authority some time ago.

The new contract covers construction of the 5,680-ft. subaqueous section of the north tunnel and an 800-ft. rock tunnel under King's Bluff, Weehawken; ventilation shafts in New York and New Jersey and various incidentals are also covered. The contract does not cover furnishing and placing the side-wall tile, electrical installation, or the roadway pavement in the tunnel.

The river portion of the north tunnel will be built, as was the south tunnel, by the shield method under compressed air. The tunnel will have an outside diameter of 31 ft. and will carry a two-lane roadway and a raised sidewalk on one side of the tube.

Shield method to be used

The contract calls for completion of all work pertaining to the construction shaft by July 1, 1938 and for completion of the tunnel to a point which permits installation of electric wiring and equipment by July 1, 1939. It must be possible to start the tunnel finish by September 1, 1939, and all work involved in the present contract is required to be completed by May, 1940.

The south tunnel, which is now under construction, will be completed with its approaches on both sides of the Hudson River and open to vehicle traffic about January 1, 1938. It will be operated thereafter with one lane of traffic in each direction until the north tunnel is completed. The present status of the construction of the south tube of the vehicle tunnel is as follows:

The shield-driven underwater section between the New York land ventilation building and the New Jersey ventilation building is nearing completion, as are the ceiling tile, the New Jersey ventilation shaft, and the river shaft at the New York end.

The rock tunnel under King's Bluff, west of the New Jersey ventilation shaft, has been excavated, and the structural crete steel lining has been erected. The interior concrete lining is now being placed at a satisfactory rate.

The New Jersey ventilation building and the New York river ventilation building are approaching completion, and the equipment in the tunnel, the ventilation installation of mechanical and electrical shaft, and the ventilation building has been started.

Contracts have been placed for the wall tile, and interior wall finish work in the rock tunnel section in New Jersey will be started about November 1, 1937.

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McQuigg Is Engineering Dean At Ohio State University

Charles E. McQuigg, director of research for the Union Carbide & Carbon Company in Long Island City, has been chosen to become Dean of Ohio State University's College of Engineering on July 1, Since the retirement last summer of Dean Embury A. Hitchcock, Junior Dean William J. Turnbull has been acting dean of the college.

Mr. McQuigg's graduation from Ohio State University in 1909, was followed by experience with the Santa Fe Railroad and the Anaconda Copper Company and then by five years as head of the department of metallurgy at Pennsylvania State College.

During the World War Mr. McQuigg was a captain in the ordnance department and was for two years after the war in the research and development laboratories of the Electro-Metallurgical Company at Niagara. He has been with Union Carbon and Carbide since 1921.

Dean Hitchcock, now dean emeritus, was graduated from Cornell University in 1890. After two years' experience with the Corliss Engine Company, he went to Ohio State University as an assistant in the department of mechanical engineering. From 1913 to 1920 he was away from the college serving as power engineer with the E. W. Clark Management Corp. and later as vice-president of the Bailey Meter Company of Cleveland. He became dean of the college of engineering upon his return to the university in 1920.

Triborough Authority Powers Increased

New York state legislature empowers bridge authority to increase bonds and combine revenues from all projects

An act passed by the New York State legislature and signed by Governor Lehman, gives to the Triborough Bridge Authority the right to build parks and approach roads in connection with the Triborough bridge and the proposed Whitestone Bridge, about 5 miles to the eastward, increases the limit of bonded indebtedness of the authority by $3,000,000, and permits pooling the income from the tolls of both bridges. (The War Department hearing on the Whitestone Bridge last week indicated that approval would be granted as soon as some changes were made in an approach bridge over Flushing Creek.)

The act provides that the authority may, with the consent of the city of New York, build parks and approach roads in the vicinity of the Triborough and Whitestone Bridges. The land for the parks is to be acquired at the expense of the city, and the parks will be maintained after construction by the city park department. In order to pay for the construction of such parks and approach roads, the provision of the original act creating the Triborough Bridge Authority which required that, as soon as the bonds issued for either of the bridges had been paid off from tolls, ownership of the bridge must revert to the city has been changed so that the Authority may continue to collect tolls on the bridges until such time as all its obligations, including those for parks and approach roads, shall have been paid off. Moreover, the provision that the tolls from each of the bridges shall be dedicated exclusively to the charges for that bridge has been changed to provide that toll collections on either of the bridges may be applied, after the

(*Continued on page* 243)

NEXT SUSPENSION BRIDGE OVER EAST RIVER AT NEW YORK CITY, IF—

the War Department grants a pending permit and if the Triborough Bridge Authority can sell the bonds to defray its $18,000,000 cost. It is the Whitestone bridge, 5 miles up-river from the present Triborough crossing. A main span of 2300 ft. and side spans of 735 ft. will comprise the main bridge, which will provide a four lane roadway 135 ft. above the river. Long viaduct approaches will be used in both the Bronx and Queens.

Illinois Engineers Meet in Chicago

Meeting of Illinois Society of Engineers covers wide range of topics in municipal and sanitary fields

Ground water recession, rental financing of sewerage work, and building-line regulation providing for future street widening, were high points at the annual meeting of the Illinois Society of Engineers held in Chicago Jan. 29-30. Simultaneous sectional meetings were arranged to care for the overflowing program to be handled in three business sessions. In this way the program was disposed of according to schedule but with limited attendance at some of the sectional meetings. The Illinois Award, a medal for the best paper read in 1936, was presented to S. M. Wood for his paper on the cycles of the Great Lakes. The president for 1937 is W. D. Jones of Chicago; vice president, W. B. Walraven, Springfield, Ill.; secretary, H. E. Babbitt, Urbana, Ill. The 1938 meeting will be held at Peoria, Ill.

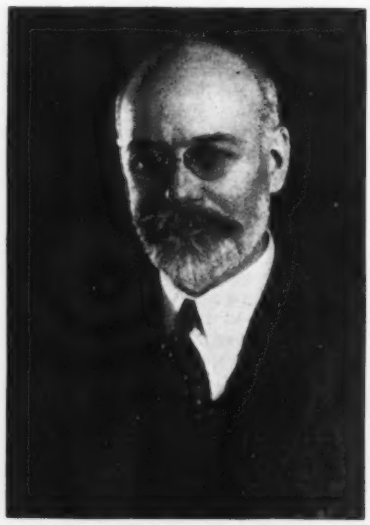
Methods of financing sewerage work were the subject of much discussion at the meeting of the Sewerage Section. Until recent years, sewerage works in Illinois were mainly financed by general bonds or special assessment, but there is now an increasing use of sewerage revenue bonds, issued under a new law and retired by service charges, or so-called rentals, paid by the users of the sewer system. Of 49 municipal plants, built since 1934, 31 were financed by this plan, and, of these, eight used the combined sewer and water revenue, providing for the handling of the water from its production and distribution to its collection, treatment, and final disposition as waste. These charges are based either on water consumption, sewer connections, or number of fixtures.

Continued recession of ground water levels in both drift and rock wells in Illinois, averaging 1.51 ft. per decade and reaching a maximum of 50 ft., may result in serious shortage, according to W. D. Gerber of the Illinois State Water Survey. Increasing demand for ground water, the use of large volumes of water for air conditioning and cooling purposes, and the drilling of an excessive number of wells, often at haphazard, is causing a serious draft on adjacent resources, which are then slowly replenished by flow from more distant areas, causing an increase in pumping head and consequently in operating costs. It appears probable that Illinois will follow other states in adopting groundwater legislation.

Bond issues are not economical for the development of a county system of secondary roads in the opinion of R. M. Lobdell, superintendent of highways for Lake County. Mr. Lobdell also held that it is better to reduce mileage than to cheapen the construction of the roads. Since county boards are usually disinclined to commit themselves to long-term plans, he advocated a pay-as-you-go plan extending over not more than six years. Street widening to give increased traffic capacity was reviewed by E. O. Pearson, assistant city engineer of Peoria, who referred particularly to the merits of building line regulations to prevent the erection of costly structures within future street lines. Such regulations, he said, have been upheld by the U. S. Supreme Court. Other considerations in bringing streets up-to-date include paving, intersections, parking facilities, street lighting, traffic control, and the safety of pedestrians.

Continuous girder and truss bridges are being used extensively, up to a maximum of five spans, by the Illinois State Highway Department, it was stated by Alfred Benesch, engineer of grade separation. Some steel girder bridges have been built with 115-ft. central spans and 90-ft. side spans. The largest continuous truss bridge, at Meredosia, has three spans of 500-565-500 ft., with the center span erected as a cantilever. Most rigid frame structures have been built as single spans, but a few of two or three spans have been constructed.

Aluminum alloy emergency bulkheads used at Gallipolis Dam on the Ohio River, were described by D. J. Fletcher. In each 125.5-ft. gate opening, seven bulkhead units, 129.5 ft. long and 4 ft. 4 in. deep, are used. The weight of the aluminum bulkhead is only 28 tons as compared with 44 tons for nickel steel and 78 tons for structural steel.



Desbarats Elected President By Canadian Engineers

George J. Desbarats was inducted into the presidency of the Engineering Institute of Canada on January 30 at their Montreal meeting.

Mr. Desbarats, born in 1861, attended the Ecole Polytechnique of Montreal and in 1879 entered the service of the Dominion government as assistant engineer in the Department of Railways and Canals, working in particular on the Sault, the Welland and the Soulanges Canals. Later he engaged in railway construction in British Columbia, particularly on the Canadian Pacific Ry., and in 1896 he re-entered the government service to take charge of a hydrographic survey on the St. Lawrence River. Five years later he was appointed director of the government shipyard at Sorel, and in 1909 he became Deputy Minister of Marine and Fisheries and, the next year, Deputy Minister of Naval Service. When, in 1924, the naval, military and air forces of the Dominion were placed under the Department of National Defense, Mr. Desbarats became acting deputy minister of that department, an office he held until his retirement in 1932.

Mr. Desbarats has been a member of the institute since 1897 and was elected an honorary member last year. He has four times served on the Council and is a past vice-president, besides being active in Institute and branch committee work.

Silicosis Control Problems Summarized by Conference

Four extended committee reports on silicosis problems and regulation were presented at a National Silicosis Conference held at the Department of Labor on Feb. 3.

Dr. R. R. Sayers of the U. S. Public Health Service, reporting for the medical control committee, presented an approximate rule for safe limit of dust concentration: Dust count in particles per cubic foot, multiplied by the fraction representing the silica percentage in the dust, should not exceed 5 million. As it is usually tuberculosis that disables and kills the silicotic, and at the same time tuberculosis is the chief predisposing factor to contracting silicosis, individuals with active tuberculosis are considered unfitted for further employment in even a minimal silica exposure.

Reporting on engineering control, W. A. Cook of the Connecticut State Department of Health summarized the methods that have found successful use in practice. Subjects included were plant location and design, the latter especially with respect to elimination of ledges; good housekeeping; ventilation; wet methods of control; and respirators.

Dealing with the economic, legal and insurance problems, V. J. Ahearn, of the National Sand & Gravel Association, recommended physical examination of all employees before exposure, and elimination of tubercular persons. The report also recommended that when the dust count is kept within the limit set by an official government standard there should be no liability for subsequent disabling silicosis, but disablement should be covered by compensation benefits. The accrued liability (for silicosis previously contracted) should be recognized as a public liability. Insurance of the compensation obligation should be compulsory, the committee held.

Labor representatives opposed medical examination of workers and demanded government insurance. Insurance through private companies was denounced as discriminating against elderly workers and leading to the discharge of workers found to have lung affections.

Wendt Again Heads Consulting Engineers

The council of the American Institute of Consulting Engineers announces the election of the following officers for the coming year: president, Edwin F. Wendt; vice-president, Robert Ridgway; secretary and treasurer, Philip W. Henry. Mr. Wendt has served as president during the past year.

Triborough Bridge Authority Powers Increased

(*Continued from page 241*)

fixed charges on that bridge have been met, to the payment of fixed charges on the other bridge or on the parks and approach roads.

The provisions limiting the bonded indebtedness of the authority have also been changed. Previously, the authority was restricted to a bonded indebtedness of $50,000,000, of which $35,000,000 must be for the Triborough Bridge and $15,000,000 for the Whitestone Bridge. Under the new act the authority is limited to a total indebtedness of $53,000,000 with no restrictions as to its distribution. Power is also granted the authority to refund bridge authority bonds whenever it seems desirable.

Finally, an assurance is given to the bond holders that no bridges or tunnels which would compete with the Triborough or Whitestone Bridges will be built until the Triborough Bridge Authority bonds have been retired.

Railway Bridge Authorized Across the St. John River

The federal government of Canada has authorized the Canadian National Railways to proceed with the construction of a new bridge across the St. John River in New Brunswick, at Fredericton, the capital of the Province. It is expected that the cost will be about $1,500,000.

An old nine-span structure which had served at this site for 52 years was destroyed on the 19th of March last by an ice jam created by the flood conditions then prevalent. Since that time there has been much discussion as to whether the bridge would be replaced or not since although it is of great importance as the first rail crossing of the river above the city of St. John (82 miles down stream from Fredericton), the old structure did not carry a heavy traffic.

Boston Considers Building Long Aqueduct

The metropolitan water commission of Boston and the Massachusetts state department of public health will soon call for bids on test borings along the route of a proposed 28-mi. aqueduct to carry water directly into the center of Boston from the reservoir system now under construction on the Swift and Ware rivers. If the borings indicate that construction of the aqueduct is practical, a recommendation for its construction will be made to the legislature. It is estimated that the cost of the project will exceed $15,000,000.

The pipe line would run from Boston Common to Sudbury, where it would connect with the present terminal chamber of the Wachusett aqueduct. The proposed line would bypass the Sudbury and Chestnut Hill reservoirs and bring water to the city at pressures great enough so that only a small amount of pumping would be necessary. More than 50 per cent of the water entering the city through the present reservoir system has to be pumped to boost the pressure.

LARGER AWARDS FOR THE WEEK

Two hot mill buildings, Reeves Manufacturing Co., Dover, O.	$500,000
Refinery, Barnsdall Oil and Refining Co., separate contracts, Corpus Christi, Tex.	$1,000,000
Apartment, 8801 Shore Road Building Corporation, separate contracts, Brooklyn, N. Y.	$900,000
Schools, Chicago, Ill.	$512,000
Roadwork, Cook County, Chicago, Ill.	$1,339,000
Highways, North Carolina	$620,000
Miscellaneous work on connection with Outer Drive Bascule Bridge, Chicago, Ill.	$955,000
Bridge over Albemarle Sound, County Project #116, Washington and Chowan Counties, N. C.	$1,275,000
Shield tunnel and rock tunnel sections and shafts, North Tube, Contract MHT-51, Midtown-Hudson Tunnel, New York, N. Y.	$8,764,000
Coal mine, Union Pacific Coal Co., of Omaha, Nebr., in Superior, Wyo.	$1,000,000

INDEX NUMBER

ENR Cost	1913 =100	1926 =100	ENR Volume	1913 =100	1926 =100
Feb., 1937	226.45	108.85	Jan., 1937	165	72
Jan., 1937	223.45	107.41	Dec., 1936	206	90
Feb., 1936	201.20	96.71	Jan., 1936	248	109
1936 (Av.)	206.46	99.24	1936 (Av.)	185	81
1935 (Av.)	195.22	93.84	1935 (Av.)	135	58
1934 (Av.)	198.10	95.23	1934 (Av.)	114	50

CONTRACTS
(Thousands of Dollars)

	Weekly Average Feb. 1936	Prev. 4 Weeks	Week Feb. 11 1937
Fed. Government	$2,468	$3,291	$1,414
State and Municipal	28,087	24,262	19,505
Total public	$30,555	$27,553	$20,919
Total private	10,570	15,967	12,091
Week's total	**$41,125**	**$43,520**	**$33,010**

Cumulative to date:
1936 (7 weeks) $370,028,000
1937 (6 weeks) $247,538,000

Note: *Engineering News-Record* reports waterworks and earthwork, reclamation or waterways projects of $15,000 and over in size; other public works, $25,000, industrial buildings, $40,000; other buildings, $150,000 and over. The figures above represent the volume of contracts and large force account projects throughout the U. S. that are of these minimum sizes or over.

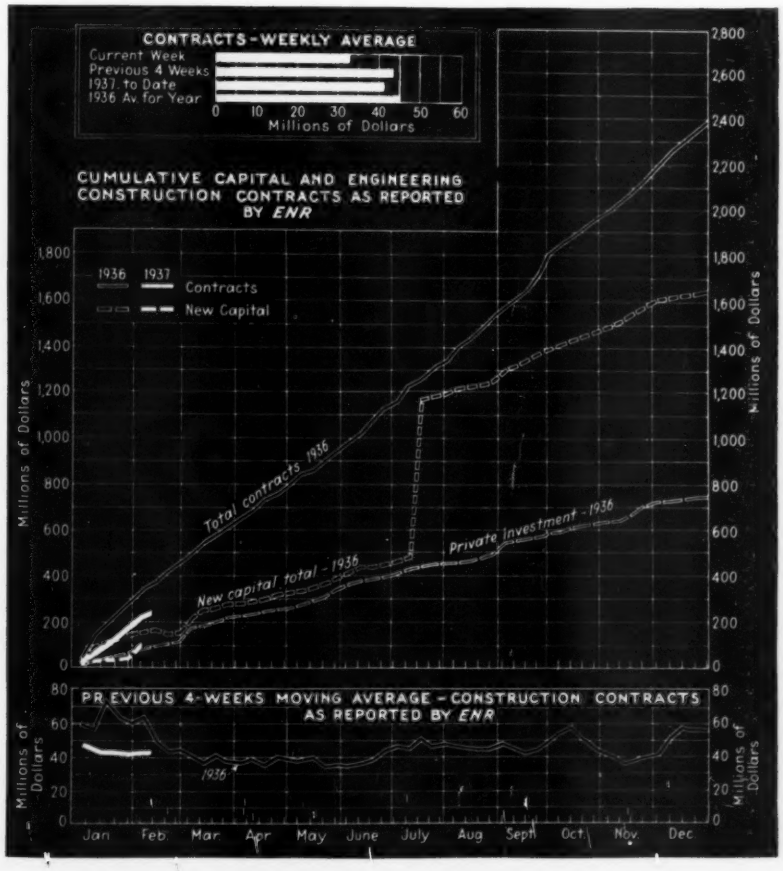
NEW PRODUCTIVE CAPITAL
(Thousands of Dollars)

1937	Week Feb. 11	Cumulative
Private investment	**$14,854**	**$118,116**
State and municipal bonds	14,854	63,113
Corporate securities	.....	55,003
Federal financing	.....	**—4,061†**
PWA non-federal Projects	.....	875
Total non-federal	**$14,854**	**$118,991**
Federal projects	.....	.....
Total	**$14,854**	**$118,991**

Cumulative to date:
1936 (7 weeks) $190,306,000
1937 (6 weeks) $118,991,000

Note: These figures include private bonds and stocks sold for new construction purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities. RFC direct purchase of bonds for "self liquidating" projects, and 25 per cent of WPA construction appropriations.

† Includes transfer from federal to private investment financing through sale by RFC of PWA bonds.



Construction Equipment and Materials

Mixer Manufacturers Hold To A.G.C. Rating

During the depression period attention of concrete mixer manufacturers has been turned to increasing the efficiency of the standardized sizes established by the Mixer Manufacturers Bureau of the Associated General Contractors. Two new types of concrete mixers were developed, the 7-S and the 10-S in discharge machines, both two and four wheel mounted. These machines were brought out by nearly all manufacturers. In addition, most manufacturers developed new designs for the 3½-S mixers, and new 28-S and 56-S machines were designed, developed, and produced by several manufacturers. In all, 50 new models and two new types of machines were introduced, but in no case was there any deviation from the A.G.C. rating.

The Mixer Manufacturers Bureau of the A.G.C., which was organized in 1924, has reduced the sizes of concrete mixers from 13 to 6 for the building industry and from 6 to 2 in the paving industry, within the range of one-yd. machines. Today, the members of this organization are producing more than 90 per cent of the mixers made in America.

New Galvanized Sheets Hold Paint

Commercial production of a new galvanized sheet that is said to assure a good paint bond on iron and steel products, has been begun by the American Rolling Mill Company of Middletown, Ohio. This new kind of galvanized sheet metal can be painted without special treatment of the surface by the user.

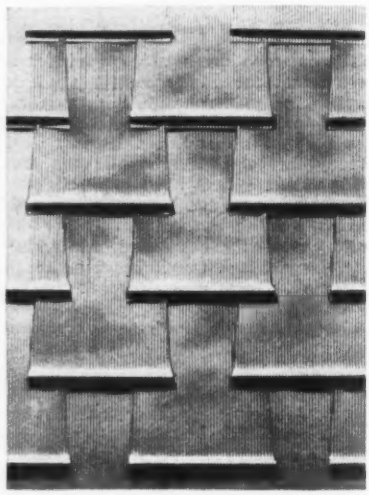
Armco Paintgrip sheets, the manufacturers say, are chemically treated to produce a finely crystalline phosphate coating which is neutral to paint and keeps the paint from direct contact with the zinc surface, eliminating the tendency of certain zinc compounds to dry the elastic constituents of paints and lacquers. The coating is an integral part of the sheet and is slightly granular in nature.

Armco Paintgrip sheets are available in any of the grades of galvanized sheets manufactured by the company. Ingot iron, plain, or copper-bearing steel may be used as the base metal.

Copper Shingles

A new type of copper shingle developed by the New Haven Copper Co. of Seymour, Conn. has a heavily corrugated surface with the corrugation lines running the length of the shingle. These corrugations take up the expansion and contraction of the shingle, insuring that it will stay tightly nailed to the roof. The shingles are built with a transverse stiffening corrugation or roll, giving a thick-butt effect which throws shadow lines and increases the strength of the roof; these rolls, it is said, also serve as snow-holders in winter. The shingle is fireproof and is said to afford good protection against lightning.

Kenmar Copper Shingles are available in four colors: red copper, copper brown,



copper green, and lead coated copper. The New Haven Copper Company recommends the shingles on any fixed roof with a slope greater than 5 in. rise per foot of run.

New Line of Welders

The Harnischfeger Corporation of Milwaukee, Wis., has developed a new line of simplified P & H Hansen "Smootharc" Welders, characterized by single current control, self-excitation, and internal stabilization.

Single current control, accomplished by a special design of shifting generator brushes, is said by the manufacturer to be the outstanding feature of this new line of welders. A simple micrometer screw shifting device permits an infinite number of current settings, while the voltage is maintained automatically. One controlling dial gives the desired welding current.

Smootharc welders are self-excited by means of a pair of auxiliary brushes placed ahead of the main brushes and connected to an auxiliary shunt winding. The shifting of both main and auxiliary brushes is accomplished by means of the special generator design, and the proper amount of excitation is said always to be available for each current setting.

Self-stabilization is secured through the use of inter-pole winding extended to the main pole shoes in such a manner that it serves the double function of inter-pole winding and stabilizing winding.

The new line of welders is built in two styles: vertically mounted in 75, 100, and 150 ampere capacity, and horizontally mounted in 200, 300, 400, and 600 ampere sizes. Both models are compactly built and streamlined to eliminate projecting parts and permit easy portability.

New Conduit is Developed for Steam and Hot Water

A new type of conduit, the Adsco-Bannon tile conduit for underground steam and hot water lines, has been developed by the American District Steam Co. of North Tonawanda, N. Y. The new tile conduit is simply designed, and is said to be easily installed at low labor cost. It consists of a substantial base drain tile supporting a circular, salt glaze, vitrefied tile conduit. The tile conduit with bell and spigot joint in about 2-ft. lengths is separable on the job into two halves along the horizontal center line. The bottom half of the conduit is placed on the base drain, the joints mortared and the trench backfilled to about the top of the half section of conduit. Pipe rollers or multiple pipe supports with rollers are placed in recesses in the two horizontal reinforcing ribs in the bottom of the pipe-support sections of the conduit.

One or more pipes can be installed, tested, inspected, and insulated with standard insulation before the top half of the conduit is placed. The top and bottom halves of the conduit are sealed with mortar in the bell and spigot joints between the projecting outside bells on each side of the conduit, and the trench is then ready for backfill.

Portable pH Meter

A new potentiometric pH meter developed by the Thwing-Albert Instrument Co. of Philadelphia, is said to be sturdy and readily portable and to be usable by anyone of reasonable intelligence. The meter, which is calibrated directly in pH units on a 4-in. dial, is graduated to 0.02 pH; the galvanometer is sensitive to 3 megohms.

The device is furnished in two models. Model 1 contains a simple potentiometric circuit consisting of a galvanometer and potentiometer and is adapted to the use of quinhydrone or antimony electrodes or both. With the quinhydrone electrode the useful range is from 0 to 8 pH and with antimony electrode it is 2 to 12 pH. Model 2, which is intended to be used with glass electrodes, has a vacuum tube potentiometer circuit and will work equally well with the quinhydrone, antimony, or glass electrode. This model, with the glass electrode, has a range of 0 to 12.5 pH. and is usable in oxidizing and reducing solutions which would poison the quinhydrone or antimony electrode.

Model No. 1 can be converted into Model 2 at low cost.

New Equipment in Brief

Lifeline Adjuster—The American Allsafe Company of Buffalo, N. Y., thinks that the Allsafe lifeline grab makes practical the use of an independent lifeline and eliminates the necessity of constantly adjusting the slack in the line. The grab

is a metal tube through which the lifeline is threaded. The ring of the safety belt is attached to the top of the grab by a forged link; as the man moves about his work the lifeline travels freely through the grab, but in case of a fall the grab immediately trips up, grabs, and locks the lifeline in tapered grooves.

Electrode—The Lincoln Electric Company of Cleveland, Ohio, has developed a new electrode, Chromeweld 4-6, intended to permit full development of the valuable properties of 4-6 per cent chrome steels.

Curve—An adjustable drawing curve manufactured by the Wade Instrument Company of Cleveland, Ohio, is made of metal-impregnated rubber with a lead core between two spring-steel strips. It is said to take easily and hold firmly any desired curve.

Pipe—Alco Products, Inc. of New York City is now manufacturing Alco electric-welded steel pipe in diameters as small as 20 in. Pipe is furnished in long lengths for any pressure and supplied with either spun bitumastic linings or hot asphalt coatings.

Lubricant—The Sinclair Refining Co., of New York City, has developed a new lubricant, Ten-ol, especially designed for use in diesel engines. The new lubricant is said to combine high film strength under extreme temperatures and pressures and resistance to blow-by of gases with free flow and cleanliness.

Welding Rod—A new electrode, Wilson No. 520, developed by the Wilson Welder & Metal Co., of New York City, is said to be especially adapted for use with the small transformer-type a.c. arc welder. It is said that an alternating current arc can be set up and maintained without difficulty with the new No. 520.

Meter Chart—Permochart, developed by the Permochart Corporation of New York City, is a smooth, hard-surface chart to be used on recording meters, from which time recordings may be removed with a damp cloth. The chart is of non-inflammable waterproof material which, it is said, will last about three years under normal operating conditions.

Pipe Clamp—The pipe clamp for stopping leaks in pipelines manufactured by the M. B. Skinner Co. of South Bend, Ind. has been redesigned with new specifications requiring a metal of higher tensile strength and uniform density and hardness. The company states that they are now testing every clamp manufactured under a pressure of 3,000 lb.

Small Engine—The Hercules Motor Corporation of Canton, Ohio, has added two small models to their line of engines, a 55.5 hp., and a 60 hp. 6-cylinder heavy-duty engine. The two models, QXA and QXB, are identical except for their bore, 3⅜ in. and 3½ in. respectively. The new QX engines are of the L head type, with cast iron pistons and a crankcase cast integral with the cylinder blocks.

Dragline—A new series of draglines, especially designed for long-range operation, is provided in Models 702 and 802 of the Koehring Company of Milwaukee, Wis. A long boom, proper stability, and extra wide and long crawlers are said to be outstanding features, while the exclusive Koehring swiveling boom point fairlead is claimed materially to reduce cable wear.

Safety Hat—A new safety hat is said by the Davis Emergency Equipment Company of New York City to combine the features of light weight and comfort with an unusual degree of protection. The hat, known as the McDonald safety hat, is molded of duralumin. It is provided with a molded rubber brim which is slightly flexible and is said to reduce the possibility of the hat being knocked off while working in close places.

Excavator—In Model 19-B, the Bucyrus-Erie Co., of South Milwaukee, Wis., has added a ⅝-yd. excavator to its line. The new excavator weighs only 15 tons. It possesses a travel speed of 1¾ m.p.h., a swing speed of 4¾ r.p.m., and a hoist line speed of 184 ft. per min. It is available with gasoline, electric or diesel power. The design of the new machine follows, in general, that of the Bucyrus-Erie 10-B, ⅝-yd. excavator.

Tractor—The new Model SO controlled ignition tractor of the Allis-Chalmers Mfg. Co., of Milwaukee, Wis., is intended to provide a low-compression diesel fuel tractor in the range between the 40-hp. Model KO and the 59-hp. Model LO. The new tractor is equipped with a 6-speed transmission, with a truck type gearshift which permits changing of gears without stopping the tractor. Wide track shoes are included as standard equipment.

Shovel—The Northwest Engineering Co., of Chicago, Ill., states that the outstanding characteristic of their new ¾-yd. shovel, Model 25, is fast action in all functions. The machine, which is fully convertible from shovel to crane or dragline with a simple change of booms, is similar in general design to the Northwest Models 15 and 18, but it is said to be larger and heavier throughout. The machine is regularly powered by a Wisconsin six-cylinder gasoline engine, but electric or diesel power is avalable.

Slide Rule—The new 398 slide rule developed by A. W. Faber, Inc., of Newark, N. J., possessing thirteen scales and a three-line runner, is said to cover the entire range of calculations involved in the solution of quadratic and cubic equations, determination of efficiencies of dynamos and motors, voltage calculations and voltage drops, weights and resistances of electric conductors, and conversion from watts to horsepower. Multiplication of three factors or division with two divisors requires only one movement of the slide.

Finishing Machine—The new Flex-Plane finishing machine developed by the Flexible Road Joint Machine Company of Warren, Ohio, is adjustable from 18 to 30 ft. for full width construction or from 9 to 16 ft. for half width construction. The machine screeds backwards as well as forward, and is equipped with a built-in gear box transmission having two speeds forward and two in reverse. Powered by a 15 hp. 4 cyl. engine, the machine has a belting attachment, reinforcing mesh strike-off attachment and a quick adjusting crown attachment on the screed.

Diesel Paver—The Ransome Concrete Machinery Co., of Dunellen, N. J., was one of the first paver manufacturers to offer a diesel-powered machine when, in the spring of 1935, they equipped their Model 27-E dual-drum paver with a diesel engine built by the Buda Co., of Harvey, Ill. They have now taken another step and are furnishing their Model 27-E single-drum paver powered with a Buda diesel, Model 6-LD-468, a six-cylinder engine of 468 cu.in. displacement. All wearing parts of this engine are fully sealed and protected to withstand the dusty conditions in which a paving engine must operate.

Blasting Cap Shunt—E. I. du Pont de Nemours & Co., of Wilmington, Del., has recently developed a shunt for short-circuiting electric blasting cap wires which is said to eliminate disadvantages of former types without sacrificing any advantages. It consists of a piece of aluminum foil about 2½ in. long wrapped around the bare ends of the wires, covering the wires and a portion of the insulation, so that no source of current can reach the bridge wire. This shunt, it is said, will not shake off while the caps are in transit, is easy to remove by spreading the wire, and since it is made of aluminum foil will not injure the hands of the user.

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Recent Publications

✦ *Cornell Rolling Doors* are cataloged in a new publication of the CORNELL IRON WORKS, Long Island City, N. Y.

✦ The MARQUETTE CEMENT MANUFACTURING Co., of Chicago, has published a *User's Manual* giving examples of and information about the use of high early strength cement.

✦ Methods and specifications for oxy-acetylene welding of pipe lines are presented in a pleasingly illustrated booklet *Lindewelding Pipe Lines* issued by the LINDE AIR PRODUCTS Co.

✦ A catalog of *Byron Jackson Deep Well Turbine Pumps* is issued, printed in black on aluminum, by the BYRON JACKSON Co., of Bethlehem, Pa.

✦ What appears to be an exceptionally complete catalog of types of and parts and accessories for traffic control devices has been combined with relevant engineering data and dimensions in a tab indexed loose-leaf book, *Traffic Signals and Controls*, by the CROUSE-HINDS Co. of Syracuse, N. Y.

✦ Two slide rule charts put out by the INTERNATIONAL NICKEL Co. of New York City, present in handy form the properties of nickel alloy steel and nickel cast iron.

BUSINESS SIDE OF CONSTRUCTION

January Construction Average Lower Private Work Doubles Last January

ENGINEERING construction awards in January total $173,077,000 for a four-week month. Of this amount $71,252,000 is private and $101,825,000 is public, including $12,145,000 in federal awards. For the five weeks of December construction awards totalled $266,301,000: private, $140,999,000; public, $125,302,000, of which $23,845,000 is federal work. January a year ago was a five-week month. Its record—total, $288,547,000; private, $43,086,000; and public, $245,461,000, including federal, $52,186,000,—with the high public figure is due largely to PWA set contract closing date, an abnormal influence absent in January 1937.

Compared with last month public and industrial buildings, waterworks, and sewerage show gains. Public buildings weekly average, $7,304,000, topped December by 41 per cent; industrial buildings, $10,631,000, weekly, gained 7 per cent; waterworks, $2,620,000, weekly, increased 84 per cent; and sewerage, $2,692,000, weekly, gained 70 per cent. All other weekly classified averages are below those of last month, commercial buildings, 22 per cent; bridges, 18 per cent; earthwork and drainage, 33 per cent; unclassified construction, 72 per cent; and streets and roads, 11 per cent.

The private engineering construction weekly average, $17,813,000, for January 1937, records a 107 per cent increase over last January. Industrial and commercial buildings are strong contributing factors. The industrial building weekly average, $10,631,000, is 308 per cent higher and the commercial building average, $1,423,000, is 90 per cent higher than last January. These increases over January last year are offset by losses in earthwork and drainage of 75 per cent, public buildings of 54 per cent, bridges of 21 per cent, waterworks of 27 per cent, sewerage of 43 per cent, unclassified of 19 per cent, and streets and roads of 41 per cent.

The Middle Atlantic states continued their hold on the leadership in construction volume. It is the only section in which the weekly averages gained over last month and the five-week January of last year.

ENGINEERING CONSTRUCTION CONTRACTS REPORTED BY *ENGINEERING NEWS-RECORD* IN JANUARY, 1937

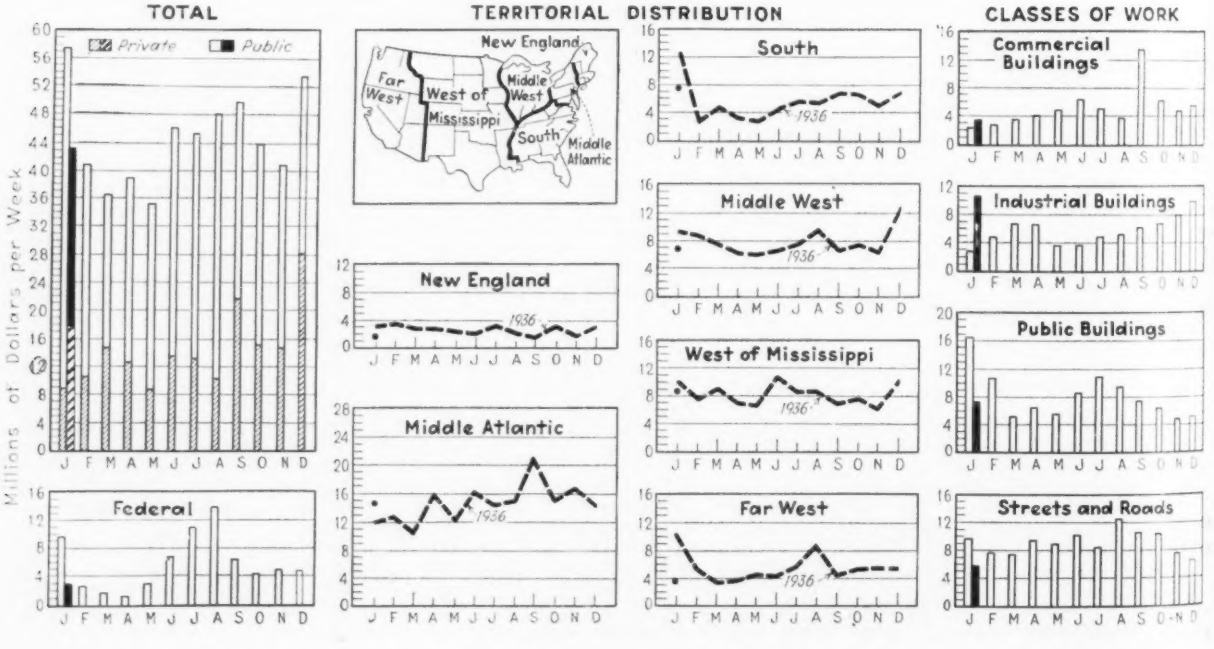
Engineering News-Record *reports projects of the following minimum costs: Water-works, excavation, drainage and irrigation,* $15,000; *other public works,* $25,000; *industrial buildings,* $40,000; *other buildings,* $150,000.

Four Weeks — Thousands of Dollars (000 Omitted)

	New England	Middle Atlantic	South	Middle West	West of Mississippi	Far West	United States January 1937	United States January 1936	Canada
Public Works									
Waterworks	377	6,478	690	584	1,226	1,125	10,480	17,941	100
Sewerage	59	3,887	949	3,293	1,752	829	10,769	23,391	55
Bridges, public	1,760	2,464	1,766	1,162	1,545	1,887	10,584	19,833	2,671
Earthwork and waterways	.....	1,094	53	532	6,766	473	8,918	44,485	23
Streets and roads	762	3,406	4,119	5,489	6,866	2,559	23,201	49,543	285
Buildings, public	1,094	11,344	3,333	7,233	3,494	2,717	29,215	80,032	276
Unclassified, public	117	5,778	628	442	403	1,290	8,658	10,236	91
Total public	4,169	34,451	11,538	18,735	22,052	10,880	101,825	245,461	3,481
Federal government (included in above classifications)	414	3,893	1,327	354	5,221	936	12,145	52,186	.....
Private									
Bridges, private	134	467	837	20	307	.....	1,765	958	.....
Buildings, industrial	1,126	10,869	17,159	5,482	5,214	2,673	42,523	13,031	432
Buildings, commercial	750	11,760	730	2,089	1,189	1,175	17,693	11,669	240
Unclassified, private	575	1,194	299	906	6,043	254	9,271	17,428	150
Total private	2,585	24,290	19,025	8,497	12,753	4,102	71,252	43,086	842
Total Engineering Construction:									
January, 1937 (4 weeks)	6,754	58,741	30,563	27,232	34,805	14,982	173,077	.....	4,323
December, 1936 (5 weeks)	16,908	70,684	35,222	63,829	53,150	26,508	266,301	.....	3,821
January, 1936 (5 weeks)	16,748	59,816	62,154	48,334	50,082	51,413	.....	288,547	12,103

ENGINEERING CONSTRUCTION REPORTED BY *ENR* WEEKLY AVERAGES

□—1936 ■—1937



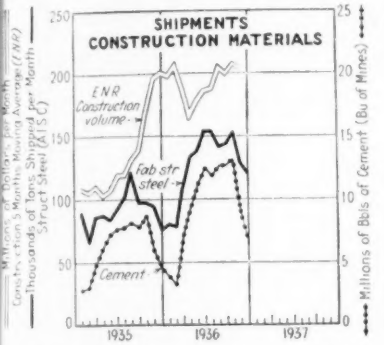
South, the only other section to show a gain, reports an increase of 8 per cent over last month.

New Capital

New capital for construction purposes totals $45,466,000 for January; state and municipal bonds, $42,259,000; corporate security issues; and PWA allotments, $875,000. The shift from federal to private financing so noticeable the latter part of last year, continues. The Reconstruction Finance Corporation sold $4,936,000 in PWA bonds to private investors during the month.

Materials

Portland Cement—The Portland cement industry in December 1936, produced 8,971,000 barrels, shipped 6,246,000 barrels from the mills, and had in stock at the end of the month 22,842,000 barrels. Production and shipments of Portland cement



in December 1936, showed increases of 54.6 and 38.4 per cent, respectively, as compared with December 1935. Portland cement stocks at mills are 0.5 per cent lower than a year ago, according to reports of the Bureau of Mines.

The ratio of production to capacity for December is 40.3, compared to 50.9 in November and 25.6 in December a year ago.

Production and shipments of cement in 1936 showed substantial gains in every section of the country. Production for the year, reported by the Bureau of Mines is 111,815,000 barrels, compared to 76,742,000 barrels in 1935. Shipments for 1936 totalled 111,977,000 barrels; for 1935, 75,233,000 barrels.

Steel—The operating rate of steel companies having 98 per cent of the steel capacity of the industry who report to the American Iron and Steel Institute averaged 79.2 per cent for January 1937, 77.5 per cent for December, compared with 49.6 per cent for January 1936.

New business booked by the structural steel fabricating industry during December reversed the seasonal trend and proved to be the largest volume of the year with the exception of July. The volume of new business totalled approximately 71.2 per cent of normal (normal being the yearly average for the four years 1928-31).

Shipments of fabricated structural steel are seasonal and December shipments are lower than the average for the year, totalling approximately 52 per cent of normal. Bookings for last year are 51 per cent larger, and shipments about 41 per cent larger than last year. The estimated total tonnage reported by the American Institute of Steel Construction is: bookings, 1,609,016 tons in 1936, 1,068,603 in 1935; shipments, 1,548,205 tons in 1936, 1,095,216 in 1935.

Lumber—In January, the lumber industry averaged 44 per cent of 1929 production, and 50 per cent of 1929 shipments, according to reports of the National Lumber Manufacturers Association. Production continued low, largely seasonal. Shipments were restricted due to the maritime strike.

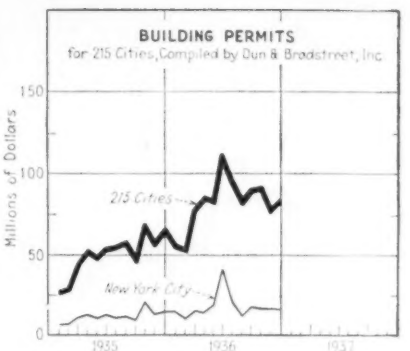
Increases in lumber prices are reported in Atlanta, Birmingham, Chicago, New York, St. Louis and San Francisco.

Building Permits

Permits for December for the 215 cities regularly reporting to Dun and Bradstreet, Inc., reached $83,562,063, representing the largest December total in six years, and a gain of 32.7 per cent above the corresponding 1935 figure of $62,992,039. The gain over the November aggregate of $75,506,623 amounted to 10.7 per cent. New York City with a total of $16,363,201 for December showed a drop of 81 per cent from November, but an increase of 15.2 per cent above December, 1935.

For the year, volume of building permits in 215 cities rose to $980,501,615, a gain of 64.3 per cent over the 1935 total of $596,686,708, and more than three times the 1933 total, $313,676,276. Every geographical section of the country shows an increase over last year.

Seasonal recessions in employment and pay rolls from November to December are shown in reports received by Bureau of Labor Statistics, Department of Labor, from 9,697 private building contractors employing 111,565 workers in December. Employment decreased 8.0 per cent during the month and pay rolls declined 9.0 per cent. Comparison of employment and pay rolls in December 1936 with December 1935 shows increases of 31.0 per cent in employment and 52.6 in pay rolls over the year interval. Reports cover employees engaged on erecting, altering, and repairing private buildings, and do not include employees engaged on projects financed by the Public Works Administration, Reconstruction Finance Corporation funds,



or regular appropriations of the federal, state and local governments.

Employment on projects of the Works Program is in excess of 2,951,000 in December, a decrease of 180,000, compared with the number employed in November. Decreases in employment occurred on projects operated by Federal agencies and on projects operated by the Works Progress Administration. More than 2,593,000 workers are engaged on projects operated by the Works Progress Administration.

The 20-cities wage average compiled monthly by the *Engineering News-Record* is higher in the skilled group, due to an increase in the structural iron workers rate. Common labor showed no change. The chart illustrating these wage average trends is now published in the wage rate section of Current Prices of Construction Materials.

The Hod Carriers, Building and Common Laborer's Union plans to notify Salt Lake City contractors of a new wage scale affecting 50 per cent of the common laborers in the city. New rates call for 65c. an hour, an increase of 15c. over the prevailing wage rates for common labor. Jackhammer operators are asking 75c. per hour in private employment, and $1.00 per hour under PWA or WPA employment.

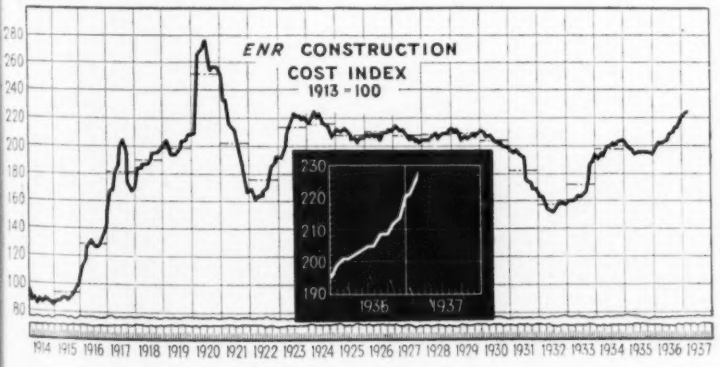
ENR Cost and Volume Index

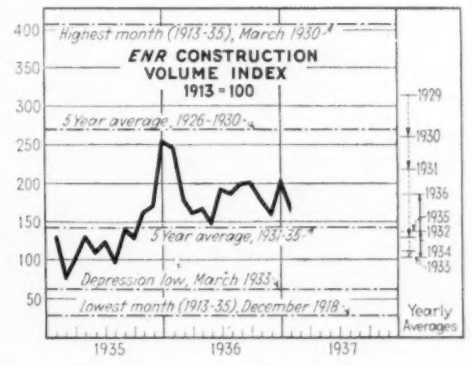
THE *Engineering News-Record* Construction Cost Index rises to 226.45 in February, the highest value since February, 1921. This rise is due to an increase in price of lumber.

The *ENR* Volume Index for January is 165.

INDEX NUMBER

ENR Cost	1913 = 100	1926 = 100	ENR Volume	1913 = 100	1926 = 100
Feb., 1937	226.45	108.85	Jan., 1937	165	72
Jan., 1937	223.45	107.41	Dec., 1936	206	90
Feb., 1936	201.20	96.71	Jan., 1936	248	109
1936 (Av.)	206.46	99.24	1936 (Av.)	185	81
1935 (Av.)	195.22	93.84	1935 (Av.)	135	58
1934 (Av.)	198.10	95.23	1934 (Av.)	114	50





Current Construction Unit Prices

Tunnel, Yosemite Park

CONSTRUCTION of 0.5 mi. of tunnel cut with approaches and portal started in December, 1936, with award of contract by the U. S. Bureau of Public Roads, C. H. Sweetser, District Engineer, to Shoiner & Gordon and Hinman Bros. Const. Co. of Los Angeles, Cal. The work forms part of the improvements on the Big Oak Flat Road, Mariposa County, Yosemite National Forest.

The unit prices and totals follow for the three low bidders: (A) Shoiner & Gordon and Hinman Bros. Const. Co. (contract) $392,718; (B) Case Construction Co., Alhambra, Calif., $443,890; (C) Walsh Const. Co., Oakland, Calif., $533,191.

	A	B	C
1. Clearing — 0.6 acre	$750.00	$500.00	$250.00
2. Grubbing — 0.6 acre	400.00	200.00	500.00
3. Mass excavation — 2,840 cu. yd	4.00	4.00	2.80
4. Struct. excavation — 166 cu. yd	4.00	9.42	4.50
5. Struct. steel — 175 lb.	.20	.25	.60
6. Cement rubble masonry — 641 cu. yd	30.00	22.50	28.00
7. Hand laid rock embankment — 705 cu. yd	10.00	4.00	4.20
8. Tunnel excavation — 2,167 lin. ft.	150.00	187.00	225.00
9. Adit excavation — 170 lin. ft.	40.00	19.00	25.00
10. Chamber excavation — 180 cu. yd.	10.00	16.00	8.50
11. Timbering — lump sum	5,000	5,000	5,000

Vermont Bridges and Highways

CONTRACTS awarded recently for the construction of 7½ miles of 18-20 ft. wide highway by the State of Vermont Highway Department and the town of Whitingham show comparisons for bituminous macadam, gravel and cement concrete surfacing as listed in the following unit bid tabulations. Also included for comparison are two 3-span overpasses and bridges, and one railroad underpass.

The locations of the projects with the dimensions, and the totals submitted by the three low bidders for each portion of the work are as follows:

I — Town of Whitingham — 0.752 mi. bituminous macadam, 18 ft.

(A) Jas. A. Farrington & Son, Whitingham, Vt. (contract)	$36,072
(B) E. A. Fuscone, Inc., Medford, Mass.	40,885
(C) T. J. Harvey & Son, Adams, Mass.	41,158

II — Wallingford — 3 span R.R. overpass and 1-beam bridge.

(A) Jos. P. McCabe, South Boston, Mass. (contract)	$90,439
(B) Lane Const. Corp., Meriden, Conn.	96,536
(C) Earle C. Hayden, Barre, Vt.	99,999

III — Brattleboro, R.R. underpass and approaches.

(A) Jos. P. McCabe, South Boston, Mass. (contract)	$49,052
(B) T. J. Harvey & Son, Adams, Mass.	54,568
(C) Chas. L. Hunter, Inc., Greenfield, Mass.	59,704

IV — Groton and Topsham, 5.2 mi. gravel m.i.p., 18 ft.

(B) Chas. E. Horne, Millbrig, Mass. (contract)	$122,651
(C) Earle C. Hayden, Barre, Vt.	125,128
(M) Lamson & Cowgill, Montpelier, Vt.	131,343

V — Waltham and Ferrisburg, 1.54 mi. cement concrete, 20 ft.

(N) D. W. Overocker, Inc., Burlington, Vt.	$68,872
(O) E. C. Hayden, Barre, Vt.	70,903
(P) Troy Paving Co., Glens Falls, N. Y.	71,877

Item	Description	A	B	C
	I — Whitingham			
1.	Rock excavation — 1,000 cu. yd.	$1.80	$2.00	$1.00
2.	Common excavation — 41,500 cu. yd.	.40	.45	.53
3.	Gravel sub-base — 3,400 cu. yd.	.80	.90	.60
4.	Patrol maintenance — 3 day	12.00	15.00	15.00
5.	Structure excavation — 140 cu. yd.	1.50	2.00	.70
6.	Broken stone base — 500 cu. yd.	4.00	5.00	5.00
7.	Bit. mac. surface — 705 cu. yd.	4.00	5.00	5.00
8.	Oil asphalt — 20,270 gal.	.13	.14	.13
9.	Concrete, cl. A — 145 cu. yd.	18.00	20.00	17.00
10.	Reinforcing steel — 19,000 lb.	.04	.05	.045
11.	Conc. rubble masonry — 15 cu. yd.	8.00	10.00	10.00
12.	Relay pipe culverts — 100 lin. ft.	.50	.50	.50
13.	Corr. galv. pipe, asph. coated, 14 in. — 8 lin. ft.	1.50	2.00	2.05
14.	Corr. galv. asph. coated, 15 in. — 60 lin. ft.	1.50	2.00	2.18
15.	Corr. galv. pipe, asph. coated, 18 in. — 260 lin. ft.	2.50	2.50	2.30
16.	Corr. galv. pipe, asph. coated, 24 in. — 38 lin. ft.	3.00	3.00	3.20
17.	Corr. galv. pipe, asph. coated, 30 in. — 44 lin. ft.	4.00	4.50	4.50
18.	Perf. corr. galv. pipe, asph. coated, 6 in. — 200 lin. ft.			
19.	Wood guard rail — 144 lin. ft.			
20.	Cable guard rail — 2,360 lin. ft.			
21.	Guide posts — 18 each			
22.	Guard rail anchors — 14 each			
23.	Fence markers — 1 each			
24.	Boundary markers — 40 each			
	II — Wallingford			
1.	Rock excavation — 600 cu. yd.			
2.	Common excavation — 48,650 cu. yd.			
3.	Gravel sub-base — 3,400 cu. yd.			
4.	Channel excavation — 10,710 cu. yd.			
5.	Structure excavation — 1,224 cu. yd.			
6.	Refined tar, dble. tank — 3,460 gal.			
7.	Concrete, cl. A — 408 cu. yd.			
8.	Concrete, cl. B — 573 cu. yd.			
9.	Reinforcing steel — 165,422 lb.			
10.	Structural steel — 360,486 lb.			
11.	Cement rubble masonry — 17 cu. yd.			
12.	C. I. pipe, 24 in. — 234 lin. ft.			
13.	Timber piles — 6,075 lin. ft.			
14.	Bank riprap — 451 cu. yd.			
15.	Perf. corr. galv. pipe, 6 in. — 60 lin. ft.			
16.	Wood guard rail — 160 lin. ft.			
17.	Cable guard rail — 4,232 lin. ft.			
18.	Cable guard anchors — 12 each			
19.	Guide posts — 1 each			
20.	Fence markers — 1 each			
21.	Boundary markers — 30 each			
22.	Slope sodding — 980 sq. yd.			
23.	Remove, reset conc. walks — 30 sq. yd.			
24.	Cement mortar, pressure — 900 sq. ft.			
	III — Brattleboro			
1.	Rock excavation — 8,000 cu. yd.			
2.	Common excavation — 17,000 cu. yd.			
3.	Gravel sub. base — 1,000 cu. yd.			
4.	Channel excavation — 1,000 cu. yd.			
5.	Structure excavation — 1,240 cu. yd.			
6.	Broken stone base — 225 cu. yd.			
7.	Conc. pavement, 1 course — 180 cu. yd.			
8.	Cr. stone M. I. P. surface — 215 cu. yd.			
9.	Refined tar — 1,025 gal.			
10.	Cutback asphalt — 4,120 gal.			
11.	Concrete, cl. A — 21 cu. yd.			
12.	Concrete, cl. C — 450 cu. yd.			
13.	Reinforcing steel — 1,900 lb.			
14.	Steel superstructure — total			
15.	Cement rubble masonry — 7 cu. yd.			
16.	Reinf. conc. pipe, 18 in. — 316 lin. ft.			
17.	Timber piles — 1,000 lin. ft.			
18.	Perf. corr. galv. pipe, asph. coated, 6 in. — 624 lin. ft.			
19.	Guide posts — 4 each			
20.	Fence markers — 1 each			
21.	Boundary markers — 20 each			
22.	Stone curbing — 900 lin. ft.			
23.	Sending sheet, etc. — 3,500 sq. yd.			
24.	Loam, top soil, backfill — 900 cu. yd.			
	IV — Groton and Topsham			
1.	Rock excavation — 2,500 cu. yd.			
2.	Common excavation — 79,500 cu. yd.			
3.	Sub-base, gravel — 22,200 cu. yd.			
4.	Patrol maintenance — 25 days			
5.	Structure excavation — 300 cu. yd.			
6.	Gravel M. I. P. surface — 3,800 cu. yd.			
7.	Refined tar — 80,800 gal.			
8.	Concrete, cl. A — 400 cu. yd.			
9.	Reinforcing steel — 23,700 lb.			
10.	Cement rubble masonry — 74 cu. yd.			
11.	Reinf. conc. pipe, 15 in. — 164 lin. ft.			
12.	Reinf. conc. pipe, 18 in. — 1,000 lin. ft.			
13.	Reinf. conc. pipe, 24 in. — 92 lin. ft.			
14.	Relay pipe culverts — 200 lin. ft.			
15.	Corr. galv. pipe, asph. coated, 15 in. — 48 lin. ft.			
16.	Corr. galv. pipe, asph. coated, 18 in. — 508 lin. ft.			
17.	Corr. galv. pipe, asph. coated, 24 in. — 504 lin. ft.			
18.	Corr. galv. pipe, asph. coated, 30 in. — 144 lin. ft.			
19.	Perf. corr. galv. pipe, asph. coated, 6 in. — 1,300 lin. ft.			
20.	Wood guard rail — 4,840 lin. ft.			
21.	Cable guard rail — 1,264 lin. ft.			
22.	Cable guard anchors — 10 each			
23.	Guide posts — 82 each			
24.	Fence markers — each			
25.	Boundary markers — 140 each			
	V — Waltham and Ferrisburg			
1.	Rock excavation — 800 cu. yd.			
2.	Gravel sub-base — 13,800 cu. yd.			
3.	Common excavation — 11,100 cu. yd.			
4.	Patrol maintenance — 8 days			
5.	Concrete pavement, 1 course — 3,515 cu. yd.			
6.	Concrete, cl. A — 40 cu. yd.			
7.	Reinforcing steel — 2,500 lb.			
8.	Cement rubble masonry — 36 cu. yd.			
9.	Reinforced conc. pipe, 18 in. — 82 lin. ft.			
10.	Relay pipe culverts — 100 lin. ft.			
11.	Corr. galv. pipe, asph. coated, 15 in. — 214 lin. ft.			
12.	Corr. galv. pipe, asph. coated, 24 in. — 80 lin. ft.			
13.	Perf. corr. pipe, asph. coated, 6 in. — 200 lin. ft.			
14.	Wood guard rail — 312 lin. ft.			
15.	Guide posts — 8 each			
16.	Fence markers — 1 each			
17.	Boundary markers — 40 each			

Current Construction Unit Prices

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	A	B	C
1. Clearing — 0.6 acre	$750.00	$500.00	$250.00
2. Grubbing — 0.6 acre	400.00	200.00	580.00
3. Misc. excavation — 2,450 cu. yd	3.00	4.00	2.80
4. Struct. excavation — 168 cu. yd	5.00	9.45	4.50
5. Struct. steel — 175 lb	.20	.25	.60
6. Cement stone masonry — 621 cu. yd	30.00	22.50	28.00
7. Hand laid rock embankment — 702 cu. yd	10.00	4.00	4.20
8. Tunnel excavation — 2,167 lin. ft	159.00	187.00	228.00
9. Adit excavation — 170 lin. ft	40.00	19.00	23.00
10. Chamber excavation — 180 cu. yd	10.00	10.00	8.50
11. Timbering — force acct	5,000	5,000	5,000

Vermont Bridges and Highways

CONTRACTS awarded recently for the construction of 7½ miles of 18-20 ft. wide highway by the State of Vermont Highway Department and the town of Whitingham show comparisons for bituminous macadam, gravel and cement concrete surfacing as listed in the following unit bid tabulations. Also included for comparison are two 3-span overpasses and bridges, and one railroad underpass.

The locations of the projects with the dimensions, and the totals submitted by the three low bidders for each portion of the work are as follows:

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(B) B. A. Simeone, Inc., Medford, Mass.	40,883
(C) T. J. Harvey & Son, Adams, Mass.	41,158

II — Wallingford — 3 span R.R. overpass and I-beam bridge.

(D) Jos. P. McCabe, South Boston, Mass. (contract)	$90,459
(E) Lane Const. Corp., Meridan, Conn.	96,526
(F) Earle C. Hayden, Barre, Vt.	99,989

III — Brattleboro, R.R. underpass and approaches.

(G) Jos. P. McCabe, South Boston, Mass. (contract)	$49,052
(H) T. J. Harvey & Son, Adams, Mass.	54,568
(J) Chas. I. Hosmer, Inc., Greenfield, Mass.	59,704

IV — Groton and Topsham, 5.2 mi. gravel m.i.p., 18 ft.

(K) Chas. E. Horne, Millburg, Mass. (contract)	$122,651
(L) Earle C. Hayden, Barre, Vt.	125,128
(M) Lambert & George, Montpelier, Vt.	131,343

V — Waltham and Ferrisburg, 1.54 mi. cement concrete, 20 ft.

(N) D. W. Overocker, Inc., Burlington, Vt.	$68,852
(O) E. C. Hayden, Barre, Vt.	70,923
(P) Troy Paving Co., Glens Falls, N. Y.	71,877

Item	Description	A	B	C
	I — Whitingham			
1.	Rock excavation — 1,000 cu. yd	$1.80	$2.00	$1.00
2.	Common excavation — 41,500 cu. yd	.40	.45	.53
3.	Gravel sub-base — 3,400 cu. yd	.80	.90	.60
4.	Patrol maintenance — 3 day	12.00	15.00	15.00
5.	Structure excavation — 140 cu. yd	1.50	2.00	.70
6.	Broken stone base — 580 cu. yd	4.00	5.00	5.00
7.	Bit. mac. surface — 705 cu. yd	4.00	5.00	5.00
8.	Oil asphalt — 23,270 gal	.13	.14	.13
9.	Concrete, cl. A — 145 cu. yd	18.00	20.00	17.00
10.	Reinforcing steel — 19,000 lb	.04	.05	.045
11.	Conc. rubble masonry — 15 cu. yd	8.00	10.00	10.00
12.	Relay pipe culverts — 100 lin. ft	.50	.50	.50
13.	Corr. galv. pipe, asph. coated, 14 in.— 8 lin. ft.	1.50	2.00	2.05
14.	Corr. galv. asph. coated, 15 in.— 60 lin. ft.	1.50	2.00	2.08
15.	Corr. galv. pipe, asph. coated, 18 in.— 260 lin. ft.	2.50	2.50	2.30
16.	Corr. galv. pipe, asph. coated, 24 in.— 38 lin. ft.	3.00	3.00	3.30
17.	Corr. galv. pipe, asph. coated, 30 in.— 44 lin. ft.	4.00	4.50	4.50
18.	Perf. corr. galv. pipe, asph. coated, 6 in.— 200 lin. ft.	1.50	1.75	1.50
19.	Wood guard rail — 144 lin. ft.	.70	.60	.60
20.	Cable guard rail — 2,360 lin. ft.	.50	.50	.50
21.	Guide posts — 13 each	2.00	2.00	2.00
22.	Guard rail anchors — 14 each	10.00	10.00	10.00
23.	Erect markers — 1 each	12.00	10.00	15.00
24.	Boundary markers — 40 each	5.00	5.00	5.00

	II — Wallingford	D	E	F
1.	Rock excavation — 600 cu. yd	2.70	2.00	1.50
2.	Common excavation — 48,650 cu. yd	.35	.40	.50
3.	Gravel sub-base — 3,400 cu. yd	.65	.60	.75
4.	Channel excavation — 10,710 cu. yd	.48	.60	.60
5.	Structure excavation — 1,224 cu. yd	4.00	4.50	2.50
4.	Refined tar, dble. tack — 3,460 gal	.16	.15	.15
5.	Concrete, cl. A — 608 cu. yd	21.00	24.00	18.50
6.	Concrete, cl. B — 573 cu. yd	18.50	16.00	18.00
7.	Reinforcing steel — 165, 422 lb	.035	.04	.425
8.	Structural steel — 566,486 lb	.036	.039	.041
9.	Cement rubble masonry — 17 cu. yd	12.00	10.00	12.00
10.	C. I. pipe, 24 in.— 234 lin. ft	4.00	5.50	4.50
11.	Timber piles — 6,075 lin. ft	.50	.40	.80
12.	Bank riprap — 451 cu. yd	1.80	2.75	2.00
13.	Perf. corr. galv. pipe, 6 in.— 60 lin. ft.	2.00	.90	1.50
14.	Wood guard rail — 160 lin. ft.	1.00	.70	.80
15.	Cable guard rail — 4,232 lin. ft.	.60	.50	.50
16.	Cable guard anchors — 12 each	8.00	10.00	10.00
17.	Guide posts — 1 each	5.00	3.00	2.00
18.	Erect markers — 1 each	5.00	15.00	10.00
19.	Boundary markers — 30 each	3.00	4.00	4.00
20.	Slope sodding — 980 sq. yd	.50	.50	.35
21.	Remove, reset conc. walks — 30 sq. yd	1.00	1.50	1.50
22.	Cement mortar, pressure — 935 sq. ft	1.00	.75	.75

	III — Brattleboro	G	H	J
1.	Rock excavation — 8,930 cu. yd	$2.00	$1.05	$2.00
2.	Common excavation — 17,000 cu. yd	.40	1.05	.60
3.	Gravel sub. base — 1,500 cu. yd	.65	.75	1.00
4.	Channel excavation — 1,500 cu. yd	.70	.75	1.00
5.	Structure excavation — 1,240 cu. yd	1.10	1.30	1.60
6.	Broken stone base — 255 cu. yd	2.75	4.75	5.00
7.	Conc. pavement, 1 course — 180 cu. yd	14.00	13.00	16.00
8.	Cr. stone M. I. P. surface — 215 cu. yd	5.00	4.75	7.00
9.	Refined tar — 1,025 gal	.14	.15	.15
10.	Cutback asphalt — 4,120 gal	.15	.15	.15
11.	Concrete, cl. A — 21 cu. yd	15.00	20.00	21.00
12.	Concrete, cl. C — 459 cu. yd	12.00	15.00	18.00
13.	Reinforcing steel — 1,930 lb	.05	.06	.10
14.	Steel superstructure — total	6,000	6,765	7,500
15.	Cement rubble concrete — 7 cu. yd	12.00	10.00	10.00
16.	Reinf. conc. pipe, 18 in.— 316 lin. ft	2.30	2.50	2.30
17.	Timber piles — 1,000 lin. ft	.10	.40	.01
18.	Perf. corr. galv. pipe, asph. coated, 6 in.— 674 lin. ft	2.00	1.50	1.50
19.	Guide posts — 4 each	5.00	2.00	2.00
20.	Erect markers — 1 each	5.00	15.00	20.00
21.	Boundary markers — 20 each	3.00	5.00	5.00
22.	Stone curbing — 400 lin. ft	1.50	.75	1.00
23.	Seeding slopes, etc.— 5,500 sq. yd	.08	.10	.07
24.	Loam, top soil, backfill — 600 cu. yd	1.25	1.50	2.00

	IV — Groton and Topsham	K	L	M
1.	Rock excavation — 2,500 cu. yd	$3.00	$2.00	$2.00
2.	Common excavation — 79,900 cu. yd	.50	.60	.60
3.	Sub-base, gravel — 22,200 cu. yd	.80	.85	.80
4.	Patrol maintenance — 25 days	15.00	15.00	10.00
5.	Structure excavation — 300 cu. yd	2.00	2.50	2.00
6.	Gravel M. I. P. surface — 3,895 cu. yd	5.40	4.50	5.00
7.	Refined tar — 85,820 gal	.13	.135	.15
8.	Concrete, cl. A — 400 cu. yd	20.00	18.00	20.00
9.	Reinforcing steel — 33,700 lb	.05	.05	.05
10.	Cement rubble masonry — 74 cu. yd	12.00	8.00	10.00
11.	Reinf. conc. pipe, 15 in.— 164 lin. ft	1.50	1.50	2.00
12.	Reinf. conc. pipe, 18 in.— 1,500 lin. ft	2.00	1.80	2.30
13.	Reinf. conc. pipe, 24 in.— 92 lin. ft	3.50	4.00	4.00
14.	Relay pipe culverts — 200 lin. ft	.60	.50	.50
15.	Corr. galv. pipe, asph. coated, 15 in.— 48 lin. ft.	1.50	1.75	2.00
16.	Corr. galv. pipe, asph. coated, 18 in.— 508 lin. ft.	2.00	2.00	2.30
17.	Corr. galv. pipe, asph. coated, 24 in.— 504 lin. ft.	3.25	3.25	4.00
18.	Corr. galv. pipe, asph. coated, 30 in.— 144 lin. ft.	4.00	5.00	6.00
19.	Perf. corr. galv. pipe, asph. coated, 6 in.— 1,300 lin. ft	1.50	1.25	2.00
20.	Wood guard rail — 4,840 lin. ft	.60	.75	.80
21.	Cable guard rail — 1,204 lin. ft	.50	.50	.50
22.	Cable guard anchors — 10 each	10.00	10.00	10.00
23.	Guide posts — 82 each	1.25	2.00	2.00
24.	Erect markers — each	5.00	10.00	20.00
25.	Boundary markers — 150 each	7.00	4.00	5.00

	V — Waltham and Ferrisburg	N	O	P
1.	Rock excavation — 800 cu. yd	$2.50	$2.50	$2.25
2.	Gravel sub-base — 13,900 cu. yd	1.10	1.20	1.25
3.	Common excavation — 11,100 cu. yd	.60	.74	.65
4.	Patrol maintenance — 8 days	15.00	15.00	15.00
5.	Concrete pavement, 1 course — 3,515 cu. yd	11.80	11.65	12.00
6.	Concrete, cl. A — 40 cu. yd	25.00	20.00	25.00
7.	Reinforcing steel — 3,500 lb	.10	.06	.06
8.	Cement rubble masonry — 36 cu. yd	8.00	10.00	10.00
9.	Reinforced conc. pipe, 18 in.— 52 lin. ft	2.00	2.00	2.20
10.	Relay pipe culverts — 100 lin. ft	.50	.50	.10
11.	Corr. galv. pipe, asph. coated, 15 in.— 214 lin. ft.	2.00	2.00	1.75
12.	Corr. galv. pipe, asph. coated, 24 in.— 60 lin. ft.	3.00	3.50	3.30
13.	Perf. corr. pipe, asph. coated, 6 in.— 200 lin. ft.	1.50	1.40	1.30
14.	Wood guard rail — 312 lin. ft.	1.00	.80	1.00
15.	Guide posts — 3 each	2.00	2.00	2.00
16.	Erect markers — 1 each	15.00	10.00	15.00
17.	Boundary markers — 40 each	5.00	4.50	5.00

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